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Lightweight Slag Concrete*

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1. INTRODUCTION

In the 1931 Institute Proceedings (p. 925) F. A. Randall presents and interesting study of the "Economics of Light Weight Concrete in Buildings." The concretes considered in that study ranged in weight from 50 to 150 lb. per cu. ft. and one of the most important deductions reached by Mr. Randall was:

> For example, it would be worth while to pay ten cents more per cubic foot to secure a reduction from 150-lb. concrete to 130-lb. concrete in a concrete frame building thirty stories high, and a reduction to 115-lb. or 100-lb. concrete in a five story building, depending on whether the floor was of solid slab or joist construction. The savings are more favorable in the steel frame building.

The light-weight concrete field seems therefore, to be one through which a considerable economic gain may be made, and because of this, a large number of methods of producing light weight concrete have been developed during the last few years, both in this country and abroad, particularly in Germany. Other desirable features which often accompany light weight concretes are improved heat and sound insulations. The light weight concrete thus becomes a very important part of our reinforced concrete construction, particularly building construction, and more data on this material are very much needed. This report deals with one of the light weight aggregates which has been developed for the production of light weight concrete, namely, porous slag produced by a special process of water-cooling ordinary blast furnace slag.

Blast furnace slag has been used successfully as concrete aggregate for a number of years, and tests have shown that the slag concrete is is no way inferior to concrete containing other aggregates. A very complete and thorough review of our present knowledge of slag concrete is contained in a report by Committee 201, on Aggregate Specifications for the American Concrete Institute.‡ Among the subjects

*Received by the Secretary April 6, 1934.
†Research Associate Professor of Engineering Materials, Lehigh University, Bethlehem, Pennsylvania.
‡"Blast Furnace Slag as Concrete Aggregate," Journal Amer. Concrete Inst., October, 1930, *Proceedings* Vol. 27, p. 183.

discussed in this report are physical and chemical characteristics of slag, specific gravity, weight per cubic foot, voids and absorption, resistance to abrasion, crushing strength, and chemical composition of the slag. For the concrete the report includes information on the weight, absorption, bond, durability, fire resistance, compressive strength, transverse strength, wear resistance, toughness, workability and yield, and slag as fine aggregate. This report is most helpful for anybody interested in the blast furnace slag and its suitability as concrete aggregate.

In the fall of 1933 the Republic Fireproofing Co., New York, producers of the slagblok floor system, requested Lehigh University to investigate the physical qualities of concrete having the special type of porous light-weight slag mentioned above, as coarse aggregate. This slag had a considerably lower unit weight than did the air-cooled slag—47.2 lb. per cu. ft. loose measure. The chemical analysis of the slag was:

Element	SiO_2	AL_2O_3	CaO	MgO	FeO	MnO	S
Per Cent	34.70	11.74	30.68	17.76	1.20	0.95	1.26

The purpose of the investigation was to obtain reliable information as to the qualities of the concrete containing this porous slag and compare the results with tests on gravel concrete.

The investigation was made in the Fritz Engineering Laboratory in October and November, 1933.

2. PROGRAM

The original program for this investigation is summarized in the following tabulation.

Group	Kind of Aggregates	Concrete Mixes	Kind of Tests
1	Sand and Coarse Slag Sand and Gravel	Variable	Compression, Modulus of Elasticity
2	Sand and Coarse Slag Sand and Gravel	1:2½:3½	Compression, Flexure, Bond, Diagonal Tension, Modulus of Elasticity

The major part of the program was Group 2, which included tests on a number of qualities of the concrete at 7 and 28 days. The other group was considered an auxiliary test. In Group 2 the consistency of the concrete was designed for a slump of 5 to 7 inches, both for the slag and the gravel concrete.

In addition to the above outlined investigation, tests are now in progress on the durability of the 1:2½:3½ concrete mix containing slag or gravel as coarse aggregates. The durability is measured in terms of loss in weight due to repeated freezings and thawings.

3. TEST SPECIMENS

Standard 6 by 12-in. cylinders were used for all the compressive strength tests, for the determination of the modulus of elasticity, and for the tests of bond strength. The bond specimens had a ¾-in. diameter reinforcing bar embedded for the full 12 in. in 6 by 12-in. cylinders. The modulus of rupture or the flexural strength was determined on 7 by 10 by 36-in. plain concrete beams. Diagonal tension strength was secured from tests of 7 by 10 by 36-in. beams with longitudinal but no web reinforcement. Standard procedure of making, curing, and testing was followed.

4. TESTING PROCEDURE

Trial mixes were used for the determination of the water content necessary for the desired slump of the concrete. The proportioning of the concrete was done by weighing out the amount determined from the unit weight values. The mixing was done by ordinary methods in a machine mixer. The consistency of the concrete in Group 2, which included the study of the 1:2½:3½ mix, corresponded to a slump for the slag concrete between 5 and 6 in., and for the gravel concrete between 6 and 7 in. The specimens were cured in the moist room and tested at the ages of 7 and 28 days. For compressive, flexural and diagonal tension strengths, the tests were made in a 300,000-lb. capacity Olsen testing machine. The bond specimens were tested in a 50,000-lb. capacity Riehle machine. The bond at initial slip of the bar was taken at a measured deformation of 0.001 in.

The uniformity in results was very good for all the tests included in the investigation. The results of the study of relationship between the strength of the concrete and the quality of the cement paste used are presented in Fig. 1 for both the slag and the gravel aggregates. Both the slag concrete and the gravel concrete increased in strength very regularly with the increase in cement content in the paste. For 7-day tests the slag concrete was slightly greater in strength than was gravel concrete of the same quality paste. At the age of 28 days both concretes gave nearly the same results for ordinary strength, while for very great strength the gravel concrete gave slightly higher results than did the slag concrete. These results indicate that concrete of nearly any desired strength may be secured with slag as well as with gravel aggregates. Light weight aggregates often produce concrete having considerably lower modulus of elasticity than gravel concrete. This unfortunate behavior was not found for the slag concrete, the modulus being nearly the same for slag and gravel concretes of the same strength. Modulus of elasticity determination of these con-

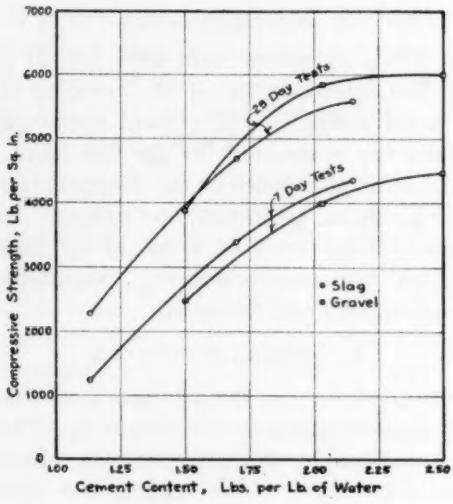

FIG. 1—7 AND 28-DAY STRENGTHS OF SLAG AND GRAVEL CONCRETE

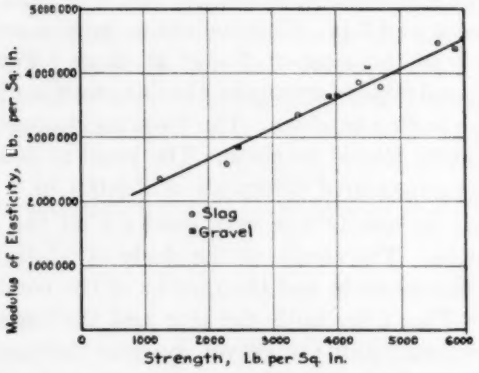

FIG. 2—RELATION BETWEEN MODULUS OF ELASTICITY AND COMPRESSIVE STRENGTH OF SLAG AND GRAVEL CONCRETE

crete mixes showed that the modulus increased regularly with the increase in compressive strength. Fig. 2 shows the modulus of elasticity plotted against the strength for both ages of test. Attention should be given to the fact that the modulus both for slag and gravel concretes, followed nearly the same trend.

The summarized results of the investigation of the 1:2½:3½ mix are given in the following table. The fine aggregate was natural sand, and the coarse aggregate was either slag or gravel.

Kind of Coarse Aggregate	Age Days	Average Results, p. s. i.				
		Compression	Initial Bond	Flexure	Diagonal Tension	Modulus of Elasticity
Slag	7	2300	404	446	193	2,275,000
	28	3740	708	545	233	3,520,000
Gravel	7	2250	438	414	183	2,430,000
	28	3760	742	554	255	3,880,000

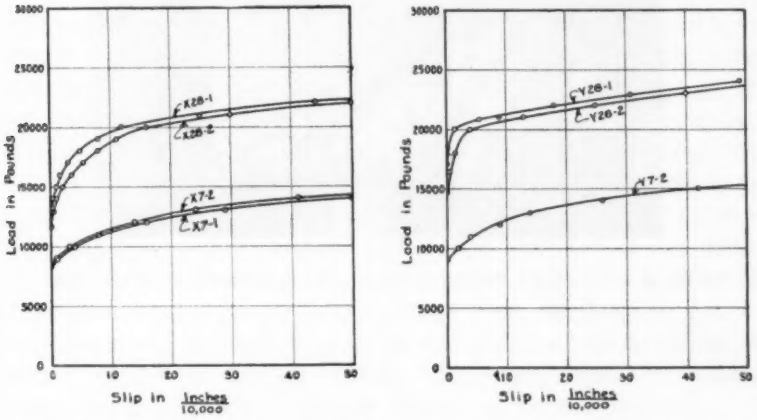

FIG. 3—LOAD-SLIP CURVES FOR BOND TESTS OF SLAG CONCRETE

FIG. 4—LOAD-SLIP CURVES FOR BOND TESTS OF GRAVEL CONCRETE

Fig. 3 and 4 give the load-deformation curves for the bond tests of slag and gravel concrete.

The diagonal tension strength was obtained on beams reinforced against failure in flexure. The type of failure given by these beams is shown in Fig. 5.

6. DISCUSSION OF RESULTS

The sand-coarse ratio had to be at least 40-60 for slag aggregates to produce a smooth and placeable concrete. With placeable mixes the results showed that the strength of the concrete was approximately the same for concrete having either slag aggregates or ordinary gravel aggregates. This was true for compressive strength, bond, flexure, and diagonal tension, as well as for modulus of elasticity. The weight of the 1:2½:3½ concrete mix having slag as coarse aggregate was approximately 127 lb. per cu. ft. in saturated condition. When dried

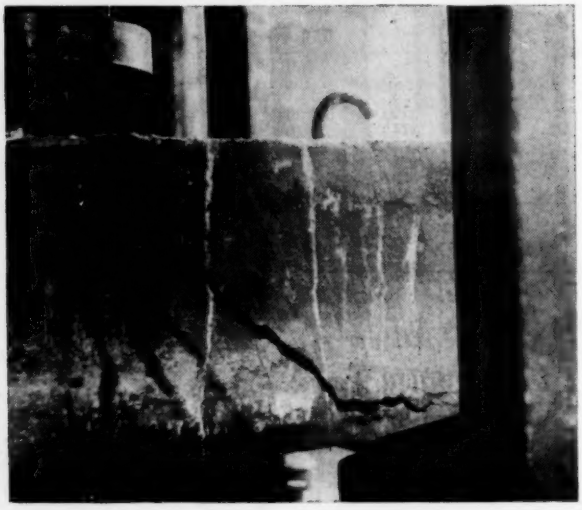

FIG. 5—TYPICAL SAMPLE OF DIAGONAL TENSION FAILURE

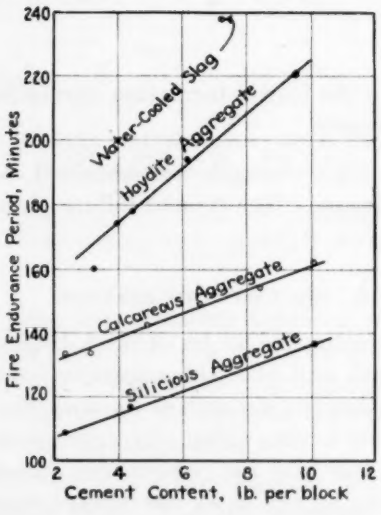

FIG. 6—EFFECT OF TYPE OF AGGREGATE ON FIRE ENDURANCE OF WALLS OF CONCRETE MASONRY (MENZEL'S DATA)

to constant weight at a temperature of approximately 100° C., the average unit weight was 113 lb. per cu. ft. The yield of the 1:2½:3½ slag concrete was about 6.00 sacks of cement per cubic yard of concrete.

The investigation showed that water-cooled light-weight slag concrete can be made of as high strength quality as that of ordinary gravel concrete.

7. INSULATION QUALITY

Carl A. Menzel in his recent report on "Tests of the Fire Resistance and Strength of Walls of Concrete Masonry Units," published by the Portland Cement Association, January, 1934, presents data helpful in judging the insulating quality of water-cooled porous slag. The study presented in Fig. 6, of effect of type of aggregate on fire resisting qualities of concrete masonry units, has been taken directly from Tables II and III of Mr. Menzel's report. Fig. 6 shows clearly how the porous light-weight concretes containing Haydite and water-cooled slag have a much higher fire resistance than have calcareous and silicious aggregates. It also shows that concrete containing water-cooled slag is considerably more fire resisting than is Haydite concrete. These data would indicate that water-cooled slag would be an excellent material for the production of heat and sound insulating concrete. Further experimentation is needed on this important problem.

8. CONCLUSIONS

The results obtained indicated that:

1. Porous slag with the proper sand content will give placeable concrete mixes.

2. The weight of the concrete having sand and coarse slag as aggregates was considerably less than the weight of ordinary concrete. The dry weight being only 113 lb. per cu. ft.

3. The yield was less for slag concrete than for gravel concrete.

4. The compressive, bond, flexural, and diagonal tension strengths of the concrete were very nearly the same for slag and for gravel aggregates.

5. The modulus of elasticity of the concrete increased with the increase in compressive strength and was approximately the same for slag concrete and for gravel concrete.

6. Menzel's fire tests showed an excellent fire resisting quality of light weight concrete having water-cooled slag aggregate.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for March-April 1935. Discussion should be available to the Secretary by Feb. 1, 1935.*

Discussion of a paper by A. G. Timms and N. H. Withey:

"Temperature Effects on Compressive Strength of Concrete"*

D. S. MacBride (*New York*—by letter): A noteworthy contribution has been made in this paper to our knowledge of the behavior of modern cements at subnormal curing temperatures. These data will prove, I believe, of great value to those doing concrete work during the winter months.

By a skillfully planned program of preliminary curing, many of the conditions affecting the early life of winter-placed concrete are approximated. Thus heated concrete, without protection is simulated in the 70° series exposed to low temperatures after six hours. Sound job practice is reflected in results from specimens cured at 70° for one day using high early strength cement and three days employing the laboratory mixture. The need for longer curing periods at 70° for both cements is clearly indicated prior to exposure to such a subfreezing temperature as 16°. In a like manner spring and fall conditions are reproduced in the 50° series.

With the termination of preliminary curing at controlled temperatures, a skillful attempt has been made to reproduce several typical job situations. Due to the infinite number of temperature combinations probable on actual construction, more than the few typical series included in these tests would prove impractical.

One important factor is the interpretation of these data. I refer to the residual heat in the concrete when controlled temperature curing ends, and to the building up of temperatures in protected concrete during the early periods when heat is generated in substantial volume, by the hydration of the cement. Naturally, the internal temperature of a considerable mass of concrete will be different from that of a 3 x 6-in. cylinder, even under similar curing conditions.

I believe that the benefits to be derived from conserving the initial heat and the heat of hydration, are not fully appreciated. Some believe that the heat held in the members of, let us say, a skeleton

*Journal Amer. Concrete Inst., Jan.-Feb., 1934; *Proceedings* Vol. 30, p. 159. Paper presented 30th Annual Convention by H. F. Gonnerman, Manager of Laboratory, Portland Cement Assn.

frame building is so slight that its conservation is impractical. I have no data based upon building members but evidence taken from road slabs may prove interesting. A road slab, laid upon a cold subgrade, is more exposed to heat losses than any concrete mass I have observed. Expressed in terms of a ratio between area and volume, a road slab is 12 times more vulnerable than is a 16 x 16-in. building column, yet very definite benefits may be secured from the protection of a concrete road slab with so simple a covering as straw.

Consider a road laid in the suburbs of Cincinnati, O., in the winter of 1932-1933, using a high early strength cement. Concrete was placed upon a de-frosted subgrade which had a temperature close to freezing the greater part of the time. The temperature of the concrete when placed ranged from 60° to 75°, averaging 65°. Wet burlap and 8 to 10 in. of straw were applied promptly after finishing. The following observation is typical of many made upon this project: concrete having a temperature of 60° when covered, (atmospheric temperature 35°), had that same temperature (60°) the following morning even though the air temperatures had fallen to 15°—a drop of 20° during the night. Cores cut by the Ohio Highway Department just prior to testing indicated the following compressive strengths:

Age in Days	Ultimate Strength p.s.i.
1	1716
2	2858
3	3760
7	4630
365	5205

Two examples taken from the many paving operations in progress during the present winter bear out these observations:

First consider a NRM job, under Indiana State Highway supervision on the Lafayette Pike at the Indianapolis City Line. A 1:2:3 mix, high-early-strength-cement concrete was used; concrete placed between Dec. 3 and Jan. 20; specifications required a 550 lb. modulus of rupture before opening to traffic; heated central-mix concrete was placed at an average temperature of 75°. Before covering with wet burlap and straw, from 10° to 15° had been lost. A typical series of temperature data are as follows:

Date	Temperature at Time of Placing		Temperature of Concrete When Covered	24 Hours		48 Hours		72 Hours Modulus Rupture
	Air	Concrete		Air	Concrete*	Air	Concrete*	
12/10	37	73	—	19	46	21	44	—
12/13	33	75	62	28	59	62	68	522
1/2	28	84	64	27	50	27	48	—
1/16	36	85	—	24	45	29	50	550
1/7	40	70	72 hours later Air 27°, concrete 47°.					

*Readings made at the surface of the concrete under the straw covering.

The practice of wetting down the straw covering so that a frozen sheet would be formed over the concrete was followed on the latter part of this job. The under side of the ice was found to be uniformly wet, the top dry and hard. Indications were that higher concrete temperatures were secured under a straw covering, which had been frozen, than under dry straw. The data are too meager to allow a definite statement on this point. Opening strengths were secured consistently within three days.

By far the most spectacular demonstration of what high early strength cement, with cold weather protection, can accomplish under severe conditions, was provided by a State Highway Project recently completed in Ohio. The first concrete was placed Dec. 5, the last Dec. 30. Freezing temperatures were encountered almost from the start, but concrete was placed continuously during the progress of the work, with the exception of one or two days when the subgrade was not ready.

Concrete having an average temperature of 70° was placed on the grade; after finishing, a covering of wet burlap and straw to a depth of 10 in. remained in place until the required beam strength (600 lbs.) was obtained.

Test beams were made from each day's pour, cured and protected with the pavement. Cores were also made for test. Results of tests on 36 beams made to Dec. 23 revealed that the required flexural strength of 600 lbs. for opening pavement to traffic was obtained in a number of instances in 48 hours, the average being 72 hours. Average core test results were as follows:

Age	Lb. per Sq. Inch	No. of Tests
24 hours	1886	4
48 hours	2341	4
72 hours	3332	5
7 days	4382	6
14 days	5537	5

A particularly interesting test series was made on the concrete placed Dec. 26. The first night the temperature dropped to 7° below zero. The next morning surface readings taken under the covering indicated a concrete temperature of 44°. Air temperature at that time was 6° above zero. On the third night the air temperature dropped to 5° below zero. Test results from cores cut from that day's run are significant—24 hours, 1509 lb.; 48 hours, 2612 lb.; 72 hours, 2610 lb.

It should be noted in all instances cited that concrete temperatures were considerably higher than those observed for air. Since strengths

TABLE 1*—TEMPERATURE EFFECTS ON COMPRESSIVE STRENGTH OF CONCRETE—SUMMARY OF DATA ON PERCENTAGE BASIS FOR HIGH EARLY STRENGTH CEMENT NO. 1 (DATA DERIVED FROM TABLES 2 AND 3)

Net Water Content Gal. per Sack of Cement	Preliminary Storage in Molds Days	Per Cent of 28-Day Strength of Concrete Cured Moist at 70° F.												
		Age at Test												
		1 Day	3 Days		7 Days			28 Days				3 Months		
		Warming Period—Days												
		0	0	2	0	4	6	0	21	25	27	0	62	83
Temperature of Placing-70° F. Temperature of Exposure-50° F.														
4½	¼ at 70° F.	21	51	55	62	68	71	82	82	77	79		75	
	⅔ "		56		74	68		85	90	86			78	
	1 "		55		67	76		89	86	89			85	
	3 "				76			92	89				82	
6	¼ at 70° F.	15	37	49	55	63	63	73	78	73	65		70	
	⅔ "		46		63	66		80	77	83			74	
	1 "		52		68	70		80	85	81			71	
	3 "				67			82	82				78	
9	¼ at 70° F.	11	34	47	52	66	54	81	80	76	63		75	
	⅔ "		50		64	64		84	81	78			76	
	1 "		45		62	68		89	89	78			79	
	3 "				64			87	87				78	
Temperature of Placing-70° F. Temperature of Exposure-33° F.														
4½	¼ at 70° F.	10	31	47	44	60	56	69	73	76	64		70	
	⅔ "		39		60	78		82	88	89			83	
	1 "		47		64	77		82	94	87			80	
	3 "				73			87	91				81	
6	¼ at 70° F.	4	24	34	35	48	45	57	54	55	43		63	
	⅔ "		30		52	60		72	76	71			69	
	1 "		37		53	66		76	78	74			75	
	3 "				63			77	76				72	
9	¼ at 70° F.	4	23	37	31	60	59	61	57	67	50		58	
	⅔ "		25		49	64		77	77	75			74	
	1 "		35		49	70		72	76	75			77	
	3 "				62			76	78				77	
Temperature of Placing-70° F. Temperature of Exposure-16° F.														
4½	¼ at 70° F.	4	12	32	14	40	38	18	60	50	49			
	1 "		44		45	70		50	79	77				
	3 "				69			71	82					
6	¼ at 70° F.	2	9	28	9	28	36	12	43	30	40			
	1 "		33		36	56		37	65	60				
	3 "				60			62	70					
9	¼ at 70° F.	1	7	29	7	27	50	8	45	38	46			
	1 "		30		31	55		34	65	56				
	3 "				58			65	78					
Temperature of Placing-50° F. Temperature of Exposure-50° F.														
4½		12	43		65	76		85	89	81		92	85	79
6		8	33		54	62		76	72	71		83	72	66
9		6	30		58	62		84	74	66		90	73	70

*Tests from "Temperature Effects on Compressive Strength of Concrete" by A. G. Timms and N. H. Withey, JOURNAL of American Concrete Institute, February 1934.

(*Table I continued next page*)

TABLE I—Continued

Net Water Content Gal. per Sack of Cement	Preliminary Storage in Molds Days	Per Cent of 28-Day Strength of Concrete Cured Moist at 70° F.												
		Age at Test												
		1 Day	3 Days		7 Days			28 Days				3 Months		
		Warming Period—Days												
		0	0	2	0	4	6	0	21	25	27	0	62	83
Temperature of Placing-50° F. Temperature of Exposure-33° F.														
4½	¾ at 50° F.		33		53	61		67	71	63			73	
	1 "		41		64	69		87	87	75			86	
	3 "				58			80	77				76	
6	¾ at 50° F.		24		41	50		65	57	47			65	
	1 "		35		60	67		77	74	75			79	
	3 "				52			72	78				74	
9	¾ at 50° F.		19		38	49		37	51	49			63	
	1 "		26		49	62		82	81	61			82	
	3 "				50			74	77				70	
Temperature of Placing-50° F. Temperature of Exposure-16° F.														
4½	1 at 50° F.		14		20	54		38	66	62				
	3 "				36			53	74					
6	1 at 50° F.		9		12	37		25	47	48				
	3 "				32			40	57					
9	1 at 50° F.		6		8	39		17	52	49				
	3 "				27			37	54					

TABLE II†—SUMMARY OF DATA ON PERCENTAGE BASIS FOR HIGH EARLY STRENGTH CEMENT NO. 1 AS COMPARED TO 28-DAY STRENGTH OF LABORATORY MIXTURE

Net Water Content Gal. per Sack of Cement	Preliminary Storage in Molds Days	Per Cent of 28-Day Strength of Concrete* Cured Moist at 70° F.												
		Age at Test												
		1 Day	3 Days		7 Days			28 Days				3 Months		
		Warming Period—Days												
		0	0	2	0	4	6	0	21	25	27	0	62	83
Temperature of Placing-70° F. Temperature of Exposure-50° F.														
4½	¼ at 70° F.	21	52	57	64	71	74	85	85	80	82		78	
	⅔ "		58		76	70		88	93	89			81	
	1 "		56		69	79		92	89	94			88	
	3 "				78			95	92				85	
6	¼ at 70° F.	19	46	60	68	78	78	91	97	90	81		86	
	⅔ "		57		79	81		98	95	103			91	
	1 "		64		84	86		99	105	100			88	
	3 "				83			101	102				97	
9	¼ at 70° F.	13	41	58	64	80	66	98	98	92	77		91	
	⅔ "		61		78	79		103	99	96			93	
	1 "		55		76	83		109	109	95			96	
	3 "				78			106	107				96	

*Laboratory Mixture.

†Tests from "Temperature Effects on Compressive Strength of Concrete" by A. G. Timms and N. H. Withey, JOURNAL of American Concrete Institute, February 1934.

(*Table II continued next page*)

TABLE II—Continued

Net Water Content Gal. per Sack of Cement	Preliminary Storage in Molds Days	Per Cent of 28-Day Strength of Concrete Cured Moist at 70° F.												
		Age at Test												
		1 Day	3 Days		7 Days			28 Days				3 Months		
		Warming Period—Days												
		0	0	2	0	4	6	0	21	25	27	0	62	83
Temperature of Placing-70° F. Temperature of Exposure-33° F.														
4½	¼ at 70° F.	11	32	49	46	62	58	72	76	78	66		73	
	⅔ "		41		62	80		85	91	92			86	
	1 "		48		66	80		85	97	90			83	
	3 "				76			90	95				83	
6	¼ at 70° F.	5	30	42	43	59	56	71	67	68	53		77	
	⅔ "		37		64	74		89	94	88			85	
	1 "		46		66	81		94	97	92			93	
	3 "				78			95	94				89	
9	¼ at 70° F.	4	28	45	38	73	72	75	69	82	61		71	
	⅔ "		31		59	79		93	93	91			90	
	1 "		43		60	86		88	93	91			93	
	3 "				76			93	96				94	
Temperature of Placing-70° F. Temperature of Exposure-16° F.														
4½	¼ at 70° F.	4	13	33	14	41	40	18	62	52	51			
	1 "		46		47	72		52	83	80				
	3 "				72			73	85					
6	¼ at 70° F.	3	11	34	11	35	45	14	53	37	50			
	1 "		40		45	70		46	80	74				
	3 "				75			77	86					
9	¼ at 70° F.	2	8	35	8	33	61	10	55	46	56			
	1 "		36		38	67		42	80	69				
	3 "				71			79	95					
Temperature of Placing-50° F. Temperature of Exposure-50° F.														
4½		13	45		68	78		88	92	84		95	88	82
6		10	40		67	77		95	90	88		103	89	82
9		7	37		70	76		102	91	81		110	89	85
Temperature of Placing-50° F. Temperature of Exposure-33° F.														
4½	⅔ at 50° F.		34		55	63		70	73	66			75	
	1 "		43		67	71		90	90	78			89	
	3 "				60			83	80				79	
6	⅔ "		30		51	62		80	71	58			80	
	1 "		43		74	83		96	92	93			98	
	3 "				64			89	97				92	
9	⅔ "		23		46	60		45	63	59			77	
	1 "		32		59	75		101	99	74			100	
	3 "				62			90	94				85	
Temperature of Placing-50° F. Temperature of Exposure-16° F.														
4½	1 at 50° F.		15		20	56		39	68	64				
	3 "				38			55	77					
6	1 "		11		15	45		31	58	60				
	3 "				40			50	71					
9	1 "		8		10	48		20	63	60				
	3 "				33			45	66					

are dependent upon concrete and not upon air temperatures, it would seem that steps taken to prevent heat radiation are well worth while. I have felt for some time that winter specifications should only regulate concrete temperatures; that air temperatures are of importance only as they affect the protection required, and that they need not necessarily influence the final results.

Knowing something about the limited time available to the authors in which to prepare this material for presentation, as well as the limitations of space, I can appreciate why all of the various forms in which this material might have been assembled were not employed. Since two types of cement were used in these tests, it seems logical, however, that their relative efficiency be observed. For this purpose two tables and two graphs are presented based upon the original data as contained in the paper. The tables represent a summary in percentage form of data taken from the authors' Tables 2 and 3. Table I has been prepared in a manner similar to that employed in Table 4. Table II is similar, except that the basis used is the 28-day strength of the laboratory cement. This table is included since the engineer and the contractor are frequently required to determine the relative economy of the two classes of cement under cold weather exposure.

Of the graphs, Fig. 1 gives the age-strength relations of the two types of cement. The curves are plotted for a one day preliminary 70° storage for high early strength and one day and three days for the laboratory cement. These curing periods were selected as approximating job practice. Due to the severity of the 16° exposure, an additional curve is shown for high early strength cement reflecting 3 days 70° preliminary curing.

For exposures of 50° and 33°, one day curing of high early strength cement produces generally higher strength at all ages included, than three days curing of ordinary portland cement. At 16° the curing period should be lengthened to from 36 to 48 hours in the case of high early strength cement, and to a probable 5 days in the case of ordinary portland cement.

When cured one day at 70°, and then exposed to temperatures of 50° or 33°, high early strength cement attains at 7 days compressive strength values which are of the same order as the 28-day strength of similarly exposed ordinary portland cement having had three days preliminary curing. At 16° curing this statement holds for the 9 gal. and the 6 gal. mixtures. The 4½ gal. mix of ordinary portland cement cured 3 days shows generally higher values than the high early strength cement cured one day at 70°. However, for such extreme exposure

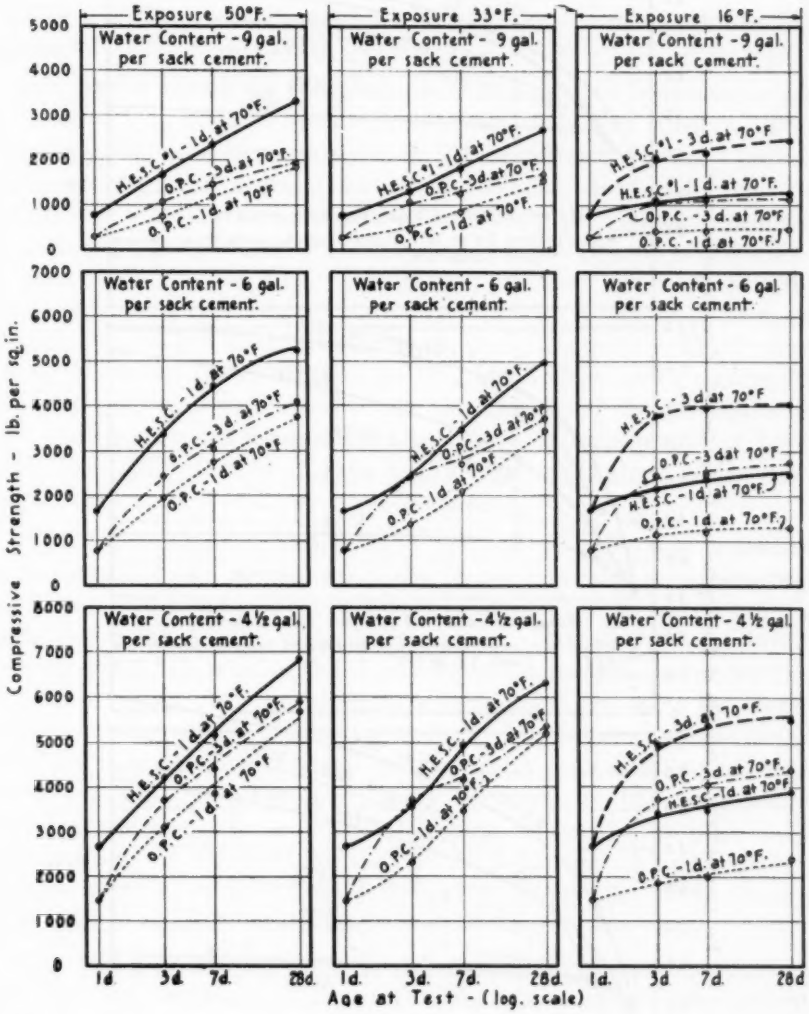
Exposure 50°F.
Exposure 33°F.
Exposure 16°F.
Water Content - 9 gal. per sack cement.
Water Content - 6 gal. per sack cement.
Water Content - 4½ gal. per sack cement.
H.E.S.C. - 1d. at 70°F.
O.P.C. - 3d. at 70°F.
O.P.C. - 1d. at 70°F.
H.E.S.C. - 3d. at 70°F.
Compressive Strength - lb. per sq. in.
Age at Test - (log. scale)
1d.
3d.
7d.
28d.

FIG. 1 — EFFECT OF EXPOSURE TEMPERATURE ON COMPRESSIVE STRENGTH OF CONCRETE MADE AT 70° F., WITH HIGH EARLY STRENGTH CEMENT NO. 1 AND LABORATORY MIXTURE OR ORDINARY PORTLAND CEMENT AND GIVEN 1 OR 3 D. PRELIMINARY STORAGE AT 70° F.—DATA FROM TABLES 2 AND 3

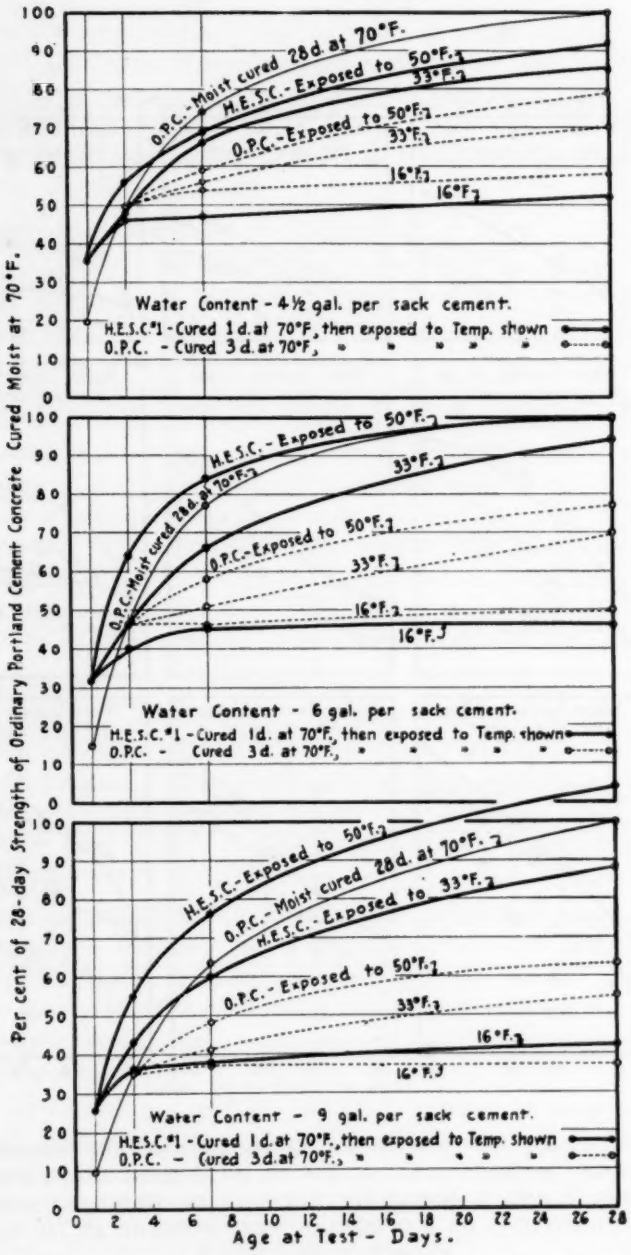
Per cent of 28-day Strength of Ordinary Portland Cement Concrete Cured Moist at 70°F.
Age at Test - Days.
H.E.S.C. - Moist cured 28d. at 70°F.
H.E.S.C. - Exposed to 50°F.
33°F.
O.P.C. - Exposed to 50°F.
33°F.
16°F.
16°F.
Water Content - 4½ gal. per sack cement.
H.E.S.C.#1 - Cured 1 d. at 70°F, then exposed to Temp. shown
O.P.C. - Cured 3 d. at 70°F, " " " " " "
Water Content - 6 gal. per sack cement.
Water Content - 9 gal. per sack cement.

FIG. 2—RELATIVE STRENGTH OF CONCRETE AS INFLUENCED BY STORAGE TEMPERATURE—DATA FROM TABLES 2, 4 AND II. PRELIMINARY STORAGE AT 70° F.—1-DAY FOR HIGH EARLY STRENGTH CEMENT NO. 1 AND 3 DAYS FOR ORDINARY PORTLAND CEMENT

curing periods longer than those just indicated are prudent to ensure satisfactory strength values. This graph clearly shows the beneficial results of three day curing for high early strength cement prior to 16° exposure.

In connection with the consideration of 16° exposure, it is interesting to observe Fig. 3 and 6 in the paper presented by the authors. The very practical conditions reflected by 70° air curing following a period of exposure to low temperature should prove of value. The relative abilities of ordinary portland cement and high early strength cement to gain strength at normal temperatures and in the absence of moisture, is shown by the following percentages: ordinary portland cement 6 gal., 3 days at 70°, 4 days at 16°, then in air at 70°, gains at 28 days a strength 36 per cent greater than its 7 day value. High early strength cement cured one day at 70°, and 6 days at 16°, gains 78 per cent over its 7-day strength. In this case the 7 day strengths of the two cements were practically the same. The strength of high early strength cement at 28 days, in this instance, is about 25 per cent higher than that of ordinary portland cement.

Fig. 2 presents the age-strength relation of these two cements both expressed as a percentage of the 28-day value for the laboratory mixture. The latter was cured 3 days at 70° while high early strength cement was cured 1 day at that temperature. This device is employed to compare on the same basis the activity of the two cements under like exposure. Note that the strength percentages decrease as the water content is increased. This is particularly noticeable in the ordinary portland cement while the high early strength cement is less affected percentagewise. Concrete made at 70° and cured at that temperature one day and then exposed to 33°, develops in the case of high early strength cement, 60 to 67 per cent at the 7-day period, and ordinary portland cement cured 3 days at 70°, 42 to 56 per cent of the 28-day moist cured value for ordinary portland cement.

This paper provides a basis for calculating the relative economy of one day's heat curing vs. 3 days and of variations in the water cement ratio in producing the desired results in cold weather using either ordinary portland or high early strength cement.

CONVENTION DISCUSSION

Answering the question whether the Portland Cement Association's investigation had indicated even tentatively any coefficient to serve as a guide in curing procedure, F. R. McMillan, Director of Research Portland Cement Association, replied: "The studies suggested recom-

mendations something like this: that concrete should be cured at 70° for 14 days (70 x 14 = 980). If the temperature falls below 70° and not below 40°, curing should be continued until the sum of the mean daily temperatures equals 980. That was a sort of rule of thumb, conforming very well with the results from these tests."

Following convention discussion by D. S. MacBride, later extended in writing as in the foregoing discussion, the question was asked whether "there was any frost in the ground when these (pavements) were laid?" to which Mr. MacBride replied: "Devices for minimizing slight penetrations of frost were developed. For example, the finishing of the subgrade was postponed as long as possible; sections of the grade which would be exposed to the night air were covered with straw; that covering was saturated with gasoline in the morning and burned. The successful road jobs I have observed have been carried on in climates where frost penetration was not severe or in periods of the year before frost penetration becomes a factor. I have yet to see a successful job carried on after frost is deep in the ground."

C. M. Morssen, (Consulting Engineer, Montreal, Canada): Were any tests made to find out whether the strength reduction of concrete cured in cold weather is temporary or permanent?

Mr. McMillan: Some specimens were exposed for instance at 16° F. and were then placed in air at 70° temperature, and we did get a considerable increase in strength.

Mr. Morssen: I had some experience in building a floor; the concrete was frozen and the building not used for two or three years, when concrete put in in cold weather was found to be just as good as the concrete put in in warm weather.

H. F. Gonnerman: In our tests strengths at 90 days indicate that after 90 days or a year, specimens cured at low temperatures closely approach strengths from normal curing, provided some moisture is available. Some years ago 6 x 12 cylinders, as soon as made, were cured at temperatures of 33, 42, 63, 121 and 212° F.; at early ages the low temperature specimens were rather low in strength compared to those cured at 63°, but at 28 days they approached the others closely.

Mr. Morssen: It would be a good point to make clear whether cold weather reduces the strength of concrete permanently or temporarily.

Mr. Gonnerman: I think that is not stressed in this paper. The tests stopped at 90 days. However, further tests are planned to show the effect of subsequent moist curing following low temperature exposure.

(Further, in response to a question, "Can concrete frozen overnight be restored by proper treatment the next day?") *Mr. Gonnerman* said: If nursed along— kept wet and warm at the same time, the concrete probably can be saved, but it should not be allowed to heat up and then freeze again. If it is frozen but once, I believe it can be brought to proper strength provided no ice crystals were formed.

(And in response to a question,) "We made no measurements on the specimens other than the strength tests reported at the various stages. We had 6000 specimens in these tests and the men assigned to them were kept busy handling these, so there was no time to make any deformation measurements.

R. J. Fuller (Consulting Engineer, Toronto): In a building in western Ontario on which I was superintendent, the first floor slab, a 1:4 concrete, was placed a little on the wet side. The temperature on the day of placing, late in December, was, throughout the day, between 40° and 45°; the concrete was placed at a temperature of 65° to 70°. A temperature of approximately 65° to 68° was maintained for about 60 hours; probably 48 hours would be a safer statement. Between 90 and 100 days later, the slab, designed for 125 lb. was subjected to a load of about 320 lb., through an error by the artificial stone man piling his stuff in the center of the slab. There was no deflection readily measured under job conditions and no sign of fracture in the slab, which went through the winter of 1926-27 from late in December. The load came on it about March 15. No particular care had been taken of the slab after the initial curing period mentioned. If I read the curves of this paper correctly, the slab should have failed, but instead of that, careful observations indicated no damage done. It seems to me we should be able to relate test conditions more closely to actual job conditions. I admit that my information is indefinite; I am talking from memory, but I have a very lively memory of the feeling of insecurity when we found that slab badly overloaded.

Mr. Gonnerman: According to Mr. Fuller's statement his concrete had 2 or 3 days protection at 65 to 70°. The Timms-Withey paper indicates that this would give strengths at the end of 3 mo. of about 80 per cent of the potential strength of the concrete. Since the load was not applied until March the concrete had gained strength in the mean- time sufficient to take the overload.

Chairman Lord suggested that the factor of safety was being lost sight of—that 70 per cent of the *ultimate* strength was probably ample for the overload.

M. A. Swayze, (Lone Star Cement Co., Inc., Hudson, New York): Mr. Gonnerman brought out one point that seems worth while to

emphasize, that we should always keep in mind the laboratory tests on cylinders. We will always have a reduction in strength when we wet up concrete and test it in the laboratory. We test concrete wet in the laboratory because it is the only way we can keep testing conditions uniform. Concrete in the field properly cured may be expected when tested dry to exceed laboratory results.

This discussion is concluded except for the authors' opportunity for closure in the November-December JOURNAL.

Discussion of the paper by R. E. Copeland and P. M. Woodworth:

"Some Tests of Load Capacity of Floors Made With Precast Concrete Joists"*

C. G. Dunnells,† (*Pittsburgh, Pa.*—by letter) Supplementing the tests reported by the authors tests were made at Carnegie Institute of Technology, Department of Building Construction, in conjunction with a study of low cost housing and residence construction. Engineers and architects employed in this survey were furnished first by the Civil Works Administration and later by the Relief Works Division of the Federal Government. The Housing Survey was under the direct supervision of Prof. Raymond W. Fisher, of the Department of Architecture, Carnegie Institute of Technology.

One of the objectives was to determine the merits of various floor systems for residence construction. The factors of comparison, that is, fire safety, structural performance and low cost, made it necessary that first-hand information be available for all types of construction considered. To obtain this information on precast concrete joists, experimental work was done and floor load tests made. These tests were more to determine the practical economy and adaptability of this type of construction to the residence field, than for a strictly technical treatment of the subject.

This investigation was made possible by the services of a junior engineer assigned from the Housing Survey to assist in the experiments. All the work was under the direct supervision of Charles W. Larkin, head instructor in masonry. Cooperation and assistance in design and testing were given by Prof. T. D. Mylrea, formerly of the Department of Building Construction, Carnegie Institute of Technology, now in charge of the Department of Civil Engineering, University of Delaware, and by Ronald W. Hadley, Portland Cement Association, Pittsburgh.

The purpose and scope of the investigations were to determine the structural safety of precast concrete joists supporting cast in place slabs and precast slabs. The several problems studied were:

*Journal Amer. Concrete Inst., Mar.-Apr. 1934; *Proceedings* Vol. 30, p. 311.
†Professor of Building Construction, Carnegie Institute of Technology.

TABLE 1—PROPERTIES OF FLOOR TEST PANELS

Compressive Tests on 6 x 12 in. Cylinders cured as test specimen, 7 days damp and 21 days air cured. Compressive strength of slab concrete not determined.

Panel No.	Proportions of Haydite Aggregate by Dry Rodded Volume Per Sack of Cement				Unit Weight of Combined Aggregates, Lbs. per Cu. Ft.	Moisure Added Per Cu. Ft. of Aggregate, Lbs.	Water per Sack Cement; Gals.	Ave. Comp. StrengthCon-crete at Time of Test, p.s.i.	Type of Reinforcement Unit	Type of Joint Between Slab and Joist
	Portion	Fine 0–6	Course 6–½	Combined Aggregate						
1–2	Joist	2.0	3.0	5.0	55	5.5	5	2960	2	Concrete Bond Metal Tie. See Fig. 1
	Slab	2.5	2.5	5.0	56	5.5	5			
3–4	Joist	2.0	3.0	5.0	55	5.5	5	3002	1	Concrete Bond Metal Tie. See Fig. 1
	Slab	2.5	2.5	5.0	56	5.5	5			
5–6	Joist	2.0	3.0	5.0	55	5.5	6.5	2490	3	Mortar Bond Metal Tie. See Fig. 2
	Slab	2.5	2.5	5.0	56	5.5	6.5			
7–8	Joist	2.0	3.0	5.0	55	5.5	7.0	2750	4	Mortar Bond Metal Tie. See Fig. 3
	Slab	2.5	2.5	5.0	56	5.5	6.5			
9–10 (a)	Joist	2.0	3.0	5.0	86		7.0	3370	4	Mortar Bond Metal Tie. See Fig. 3
	Slab	2.5	2.5	5.0	87		7.0			

(a) Aggregate for Panel 9-10 consisted of 0.4 sand and 4-½ gravel.

The value of welded reinforcement and effects of economizing the steel reinforcing unit; effect of vibration in manufacture and resulting strength; methods of bonding precast concrete slab to precast joist.

Concrete mixtures were limited to 1:2:3 for joists and 1:2½:2½ for slabs, both by dry rodded volume. Haydite, fine and coarse aggregate, was used in all panels, except panel 9-10 which was sand and gravel. Consistencies ranged from ⅝ in. to 2½ in. slump. Materials were all hand mixed and vibrated into the forms with Syntron spud type vibrator. Moisture was added to the Haydite aggregate a half hour before mixing. Difficulty was experienced in placing the concrete in the joist molds due to the dryness of the mix and the maximum size of the coarse aggregate. Vibrating the concrete produced smooth finished joists but the time required for placing discourages the use of the spud type vibrator in favor of external vibration.

Reinforcing rods were all new billet stock, intermediate grade, plain round. Reinforcing mesh for slabs was standard 6 x 6 No. 6, and No. 6 welded wire fabric. Tests were made on the load capacity and performance of floor panels for both cast in place slabs and precast slabs on precast concrete joists. Details of floor test panels are given in Table 1 and Fig. 1, 2 and 3. The reinforcement and details of assembly are shown in Fig. 4.

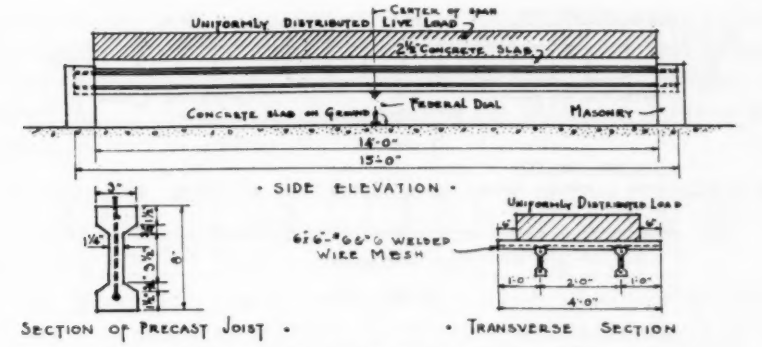

FIG. 1 – DETAILS OF PANELS 1-2 & 3-4
CAST IN PLACE SLABS ON PRECAST JOISTS

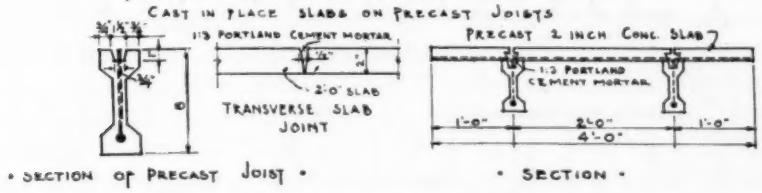

FIG. 2 – DETAIL OF PANEL 5-6
PRECAST SLAB ON PRECAST JOIST

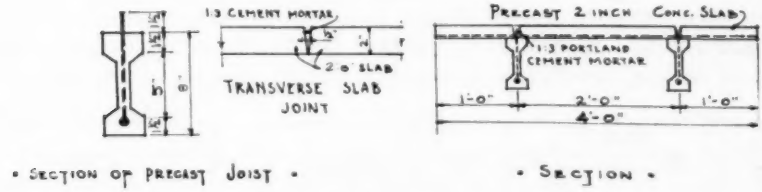

FIG. 3 – DETAILS OF PANELS 7-8 & 9-10
PRECAST SLAB ON PRECAST JOIST

FIG. 1, 2, 3

Joist reinforcement design based on the usual flexure formulas and the following factors and assumptions: Satisfactory interaction between joist and slab; $d = 8.87$ in.; $j = 0.875$; $Fs = 20{,}000$ p.s.i.; $Fv = 16{,}000$ p.s.i.; $Vs = 60$ p.s.i.; live load of 40 lb. per sq. ft.; dead load of 47 lb. per sq. ft., which includes a 16 lb. allowance for plaster ceiling and floor finish. Joists were made in steel molds and erected on masonry supports 21 days after manufacture. Cast in place slabs were supported by form work held by the joists. The slabs were cast and then covered with burlap and cured wet for 21 days. Precast slabs were cast in wooden forms and cured in the same manner. A 1:3 cement mortar, using high early strength cement, was used to

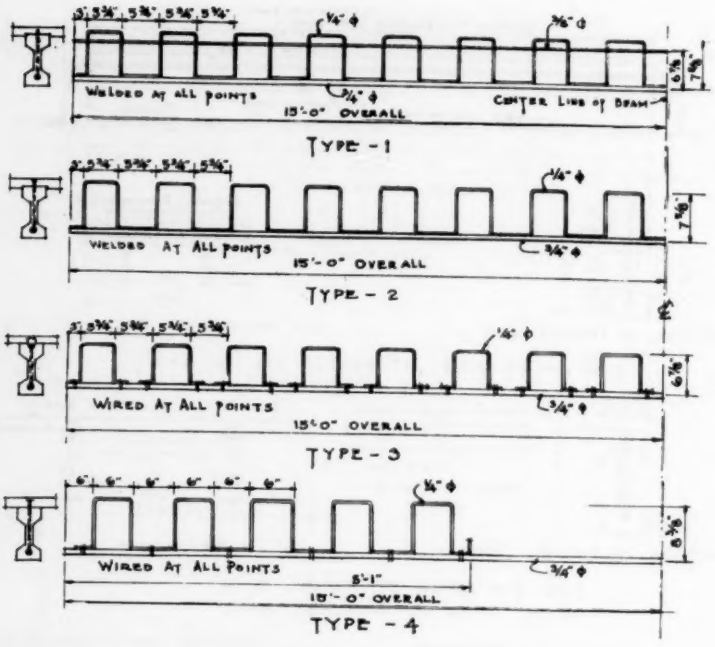

FIG. 4

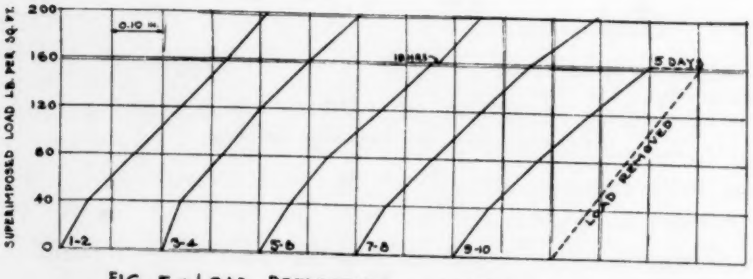

FIG. 5

bond precast slabs to precast joists. This mortar was cured wet for 7 days. Forms for cast in place slabs were stripped in 7 days. Deflection readings were made before and after the slab load was applied to joists. The mid-span deflection averaged .125 in. After initial deflection readings were made, loading of the panel with brick was started. The bricks were placed in separate tiers to prevent arching.

Deflection readings were taken after each course of brick was applied. The ultimate load was regarded as the load which produced collapse of one or both of the joists, or an evident failure as indicated by deflections. Federal dials were placed in the center of the span and deflections read to .001 in.

The test procedure for cast in place slabs on precast concrete joists was as follows. The panels were tested 21 days after the slab was cast. Fig. 5 shows Panel 1-2 loaded to 160 lb. per sq. ft.

Fine tension hair cracks developed in lower head of joist in panels 1-2 and 3-4 at loadings of 80 to 120 lb. per sq. ft. extending vertically, all in the center third of the span. With load increase, the cracks opened up and extended into the web of the joist. Several continued through the head and followed at right angles into the slab. Cracking had fully developed at loads of 200 lb. and further loading caused only increased opening of existing cracks. There was no evidence of any failure from shear or diagonal tension in either panel until after the ultimate loading was reached. There was no evidence of joint slippage at any time. Examination of the reinforcement units later indicated no failure of tension steel bond. The nature of failure indicated that it was a tension failure. In both panels failure occurred gradually evident only in the abnormal deflections indicated. There was no evidence of compressive failure in the concrete of the joist or slab in any of the panels.

The test procedure for precast concrete slabs on precast concrete joists was as follows. Panels were tested 28 days after joists were cast, 21 days after slabs were cast with slabs grouted on the joists 7 days. High early strength cement (1:3 mix) was used to bond slabs to joists. Fig. 7 shows panel 5-6 loaded to 160 lb. per sq. ft.

Cracking developed in precast slab panels at approximately the same loadings and locations as in the panels with cast in place slabs. In both panels 5-6 and 7-8 total failure occurred when the ultimate load was applied, by yielding of the steel at the section of maximum moment and by diagonal tension failure near the points of support. There was found to be no evidence of joint slippage in either panel. Examination of the reinforcement units later failed to indicate tension steel bond failure. The 160 lb. load was maintained on Panel 9-10 5 days, which resulted in increasing the deflection .106 in. On removal of the load, the panel recovered .295 in. of the .506 in. deflection.

In discussing the results of these tests we refer to Table 2, and Fig. 5 which give load capacity and deflections with increments of loading

TABLE 2—RESULTS OF FLOOR LOAD TESTS

Panels uniformly loaded with brick arranged in separate tiers to prevent arching. Load values refer to superimposed load, lb. per sq. ft. Weight of floor, 28 lb. per sq. ft. Panels 1-2, 3-4; 24 lb. per sq. ft. Panels 5-6, 7-8; 33 lb. per sq. ft. Panel 9-10.

Panel No.	Ultimate Load, Lb. Per Sq. Ft.	Ratio of Ultimate Load to 85 Lb. Live Load (a)	Mid-Span Deflections at Following Loads, Lb. Per Sq. Ft.; In.				
			40	80	160	200	240
1–2	237	3.0	.057	.147	.335	.415	1.119
3–4	257	3.2	.041	.124	.305	.405	.610
5-6 (b)	238	3.0	.061	.141	.353	.450	
7-8 (c)	240	3.0	.068	.159	.367	.498	
9-10 (d)			.073	.172	.400		

(a) Allowable live load based on FS of 20,000 p.s.i. (b) Panel received abnormal deflections when load of 238 lb. was applied. (c) Panel failed at loading of 240 lb. per sq. ft. (d) Not loaded to failure. Load of 160 lb. per sq. ft. left on panel 5 days increasing deflection .106 in. Recovery on removal of load was 0.295 in.

FIG. 6—PANEL 1-2 LOADED TO 160 LB. PER SQ. FT.
FIG. 7—PANEL 5-6 LOADED TO 160 LB. PER SQ. FT.

for all panels. Comparison of Table 2 with a like table of the results of tests by Copeland and Woodworth indicate a similarity of results for the cast in place slab panels. The panels with precast concrete slabs acted in the same manner as the cast in place slab panels. It is our deduction that the anchorage of the stirrups into the mortar joint, coupled with the anchorage of the slab reinforcement in this joint, produced a strong mechanical bond that helped materially to give the "T" beam action of the joist in the panels.

With the types of reinforcing used the variation of design produced only minor changes in the load carrying capacity of the precast concrete joists. The loadings required to produce abnormal deflections indicated ample steel reinforcement in all panels for the designed live load.

The joist concrete in every type of joist was an economical mixture. It is believed that any concrete designed for a minimum compressive strength of 2500 p.s.i. would produce satisfactory results. The slab concrete was not tested for compressive strength.

Precast slabs on precast concrete joists present a large field for further development of slab finishes. Studies will be made in vibrated slabs for smooth finished floor surfaces and ceiling effects. Decorations and molded surfaces in slabs will be developed for beamed ceiling effects in the model home of 1935, which will be constructed by the students of Carnegie Institute of Technology in the masonry laboratory. Where beamed ceilings are not desired, convenient wire hangers can be dropped through the cross joint of the precast slabs to support a metal lath and plaster ceiling. For floor finishes, attractive patterns in concrete can be made by using slabs with a colored top surface. Square panels with the mortar joints plain or in colors, would add to the attractiveness of this finish. Any other type of floor finish could be applied on precast slabs. If wood finished floors are desired, screed clips can be set in the mortar joints as it is being placed.

Precast slabs on precast concrete joists should be adaptable as an economical floor construction for residences, apartment buildings, hotels, or any structure subject to ordinary residence floor loadings. It is our desire to give these results of the tests for comparison with the tests reported by Copeland and Woodworth, and to add our results with precast concrete slabs on precast concrete joists. The conclusion is that precast concrete slabs can be used on precast concrete joists with practically parallel performance as cast in place slabs on precast concrete joists if sufficient anchorage is provided to prevent slipping of the slab when the floor is loaded.

*F. N. Menefee,** (*Ann Arbor, Mich.*—by letter) The writer notes with considerable satisfaction that the authors' results, as far as they go, check a similar study made by the writer in 1931-33. The purpose of the writer's tests was identical and the specimens for finding bond strength were exactly like those of the authors, Fig. 2, except that they (1) had no metal ties; (2) had a ¾-in. mortise or imbedment of joist into slab as compared with the authors ½-in., and (3) that the writer used several brands of cement.

The writer had experimented with shrinkage of concrete for several years and it was thought that metal ties such as used by the authors would not add sufficient strength to offset the extra cost of manufacture. What that extra cost would be is not known, but the authors tests do not indicate that the ultimate joint strength in pounds per square inch for their metal tied ½ in. mortise is any greater than a ¾ in. mortise without metal ties.

The authors' tests include only two specimens for mortised bond tests (Table 2, specimen 7) without metal ties. Hence the writer submits Tables 1, 2, 3, and 4, showing the bond strength to be expected with a ¾-in. mortise and no metal ties.

The authors obtained 340 p.s.i. bond or joint strength in their specimen 7, Table 2, with ½ in. mortise, as compared with averages of 457, 288, 373, and 339 p.s.i. as shown in the writer's tables 1, 2, 3, and 4 respectively.

In addition to the foregoing the writer made up four specimens like the authors' Fig. 2, in which the mortise was omitted entirely. The precast joist was cemented to the slabs with a mortar made of 3 lb. cement, 6 lb. sand, .43 lb. lime, and 1.55 lb. water. Table 5 shows the results. They are given to show that with careful workmanship, an approach to sufficient bond *may be had* without metal tie or mortising —not with the idea of advocating such practice. They may help to remove skepticism concerning the molded mortise joint.

In all the authors have 14 bond test specimens, in which the concrete bond was not reduced by paraffin-rosin, 12 with metal ties and 2 to concrete without. The average joint strength for the 12 with metal ties (using first 6 specimens Table 2) was 335 p.s.i. For the two specimens without metal ties, the average was 340 p.s.i. Specimen 6 ran as high as 450 p.s.i. joint strength, from which the authors, under conclusions Series A, 2, say "metal ties of any substantial type increase the joint strength remaining after the fracture of the concrete bond."

*Professor of Engineering Mechanics, Univ. of Michigan.

TABLE 1

(Jan. 1932)

Specimen	Age, Days	Mix	Cement	Bond Resistance at Failure, p.s.i.	Compressive Strength p.s.i.
1	16	1 : 2 : 3	A	432	
2	13		A	528	
3	28	1 : 2 : 4	A	482	1860
4	28		A	312	1990
5	28	1 :2 : 4	B	460	1990
6	28		B	535	1700
			B		1770
7[1]	34	1 : 2½ : 3	C	606	3270
8	34		C	547	
9	58		C	650	3060
10	58			587	
11[2]	38	1 : 2 : 4	D	346	685
12	38			269	730
13	38			305	
14[3]	38			350	

[1]This specimen did not fail in bond but by shearing of the pre-cast joist.
[2]2 lb. calcium chloride per sack of cement.
[3]4 In. silo tile replacing Lith-I-Bar pre-cast joist.

TABLE 2

(Jan. 1933)

Specimen	Age, Days	Mix	Bond Resistance at Failure, p.s.i.	Compressive Strength, p.s.i.
1	21	1 : 1.5 : 4	360	2175
2	31	"	327	2330
3	31	"	310	2450
4	31	"	180	1600
5	31	"	295	2170
6	31	"	301	2110

TABLE 3

(Feb. 1933)

Specimen	Age, Days	Mix	Bond Resistance at Failure, p.s.i.	Compressive Strength, p.s.i.
1	31	1 : 2 : 4	276	2700
2	31	"	196	
3	31	"	562	4310
4	31	"	522	3790
5	31	"	338	2870
6	31	"	355	3410

TABLE 4

(Mar. 1933)

Specimen	Age, Days	Mix	Bond Resistance at Failure, p.s.i.	Compressive Strength, p.s.i.
1	28	1 : 2 : 3	530	
2	28	"	501	
3	28	1 : 2.15 : 2.85	450	
4	28	"	210	2875
5	28	"	200	
6	28	"	130	

TABLE 5

(May 1933)

Specimen	Mix	Cement	Bond Resistance at Failure, p.s.i.
1	3 lb. sand	A	99
2	6 lb. cement	A	122
3	.43 lb. lime	B	230
4	1.55 lb. water	B	216

From a practical standpoint there would be no joint strength after the fracture of the concrete bond if there were no tie; hence conclusion 2 is without question true. It required no tests to establish it.

The very important feature of the investigation seems to have been missed. We are interested in the practical question as to whether—

(1) concrete slabs mortised over precast joist will shrink enough to grip the joist and act as a monolithic T-beam,

(2) what depth of mortise is necessary,

(3) whether or not the metal tie is necessary.

The importance of (1) and (2) is obvious; that of (3) hinges on the fact that the cost of precast joists may be increased if ties are necessary.

But the curious thing is that specimen 7 ran 5 p.s.i. stronger in bond strength than the average of the previous 6 specimens and finished in third place out of 7 entrants. It may be that specimen 7 would not repeat, that it was higher than should be expected, but by the same token specimen 6 may be high. On the strength of this specimen 6, comes conclusion 3 which says "metal ties of the type used in specimen 6 *may* be expected to increase the ultimate joint strength above that obtained with concrete bond alone." The writer agrees that it may, but is more inclined to believe that the 450 p.s.i. value for specimen 6, is due to other causes, for there is not a great deal of difference between the type of tie in specimen 6 from the type in 2 and 3, and yet 2 and 3, instead of rating higher than specimen 7 with no tie, averaged 5 p.s.i. less. Since there were only 2 specimens of 6 it hardly seems justifiable to conclude that type 6 is productive of greater joint strength than concrete bond alone.

Under the same Series A, conclusion 1 points to a range from 280 to 340 p.s.i., a very uniform set of results, but specimen 6 went to 450 p.s.i. and belongs in the same class as those which registered 280 and 340 p.s.i.

By comparing the authors Table 2, and particularly specimen 7, with the writer's Tables 1, 2, 3, and 4, it appears that by imbedding

or mortising the joist ¼ in. further than did the authors, the metal tie can be dispensed with, provided the authors observations as to mixtures are adhered to. It should be obvious that to obtain a good bond at the mortise the slab concrete must be workable and the maximum size of aggregate ¾ in. or less. The extra ¼ in. of mortise can be obtained without adding to the thickness of the floor by chamfering or rabbeting the form board adjacent the joist, as shown in the authors' Fig. 1.

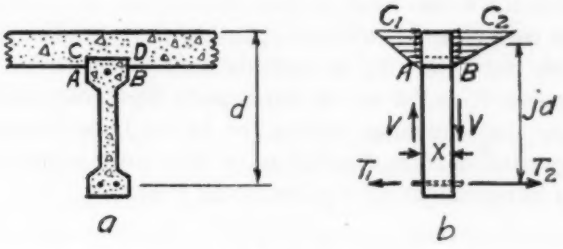

FIG. 1

In floor panels the longitudinal shear at the junction of the slab of the joist is obtained in the following way: (See Fig. 1B) By summation of horizontal forces

$$vbx = T_2 - T_1 \quad \ldots\ldots (1)$$

v being intensity of longitudinal shear, b the breadth, and x the length of the longitudinal portion of the beam under consideration. By summation of moments,

$$Vx = (T_2 - T_1)\, jd = vbxjd$$

$$v = \frac{Vx}{bxjd} = \frac{V}{bjd} \quad \ldots\ldots (2)$$

Taking the authors' panel 6, with no metal ties, the ultimate load produced 4200 lb. of shear at each end of the joist.

b = 3 in. jd = about 9 in.; from which the longitudinal shear in the joist in the plane of the bottom of the slab was about 155 p.s.i. The shear in pounds per square inch around the mortised joint will be ¾ of this, or 116 p.s.i., which is ⅓ of the ultimate shear found in specimen 7 of the authors' Table 2. These values very closely correspond to results which the writer has obtained upon panel tests exactly like the authors except that there were no metal ties, the imbeddment was at least ¾ in. and the span was 13 ft. 2 in.

In conclusion it is the writer's opinion that the authors tests substantiate the writer's belief, based upon his own tests that with

(1) properly designed joists up to 3 in. width and

(2) properly mixed workable concrete

sufficiently reinforced transverse of the joist to prevent cracking over the joist, enough shear will be developed at the joint between slab and joist to produce T-beam action similar to that obtained in monolithic construction.

In addition the writer believes that the first six of the conclusions of Series B are correct and could be applied to floor panels made without the metal tie but with a ¾ in. imbeddment. The writer also agrees with conclusion 7, as far as the laboratory test goes, but fears that specifications and drawings calling for ½ in. imbeddment with the usual wide variation from dimensions in form work would occasionally result in no imbeddment over parts of the joist.

This discussion will be concluded with the authors' closure in the JOURNAL *for November-December 1934*

Discussion of paper by Davis, Carlson, Kelly and Troxell:

"Properties of Mortars and Concretes Containing High Silica Cements"*

Thos. E. Stanton, Jr.† (*Sacramento, Calif.*—by letter): The experience of the California Division of Highways with the use of silica admixtures in concrete dates from 1929 when the Imperial Valley test series, reported in the October 1931 Journal, was started.[1]

It was not until 1929 that the addition of siliceous materials for other than improvement of workability was actively promoted in California. About this time high pozzuolanic properties were first claimed for a volcanic ash obtained from a deposit near Fresno.

Although recent practice tends towards intergrinding the siliceous material (calcined or uncalcined) with the portland cement clinker, none of the local commercial cement plants had up to that time evidenced any extensive interest in the manufacture of a modified portland cement by intergrinding pozzuolanic admixtures and the use of Fresno pumicite was therefore proposed for addition during the fabrication of the concrete.

The earlier tests in which the siliceous materials were added during the progress of the work, are not necessarily indicative of the results from intimately interground mixtures. However, the long time results of these earlier tests are of considerable interest.

In general, the conclusions reached by the authors are checked by the work of this Department both in the 1929 series and by all subsequent studies.

Because of the commercial development of hydraulic cements of the high silica type, the Materials and Research Department of the California Division of Highways is conducting an extensive series of tests of special cements of this nature, either now manufactured or proposed for manufacture locally for commercial use, but is not engaged

*Journal, Amer. Concrete Inst., Mar.-Apr. 1934, *Proceedings* Vol. 30, p. 369.

†Materials and Research Engineer, State of California, Department of Public Works, Division of Highways.

[1]Journal Amer. Concrete Inst., Oct. 1931, *Proceedings* Vol. 28, p. 131.

in any original research of laboratory manufactured products such as that described by Professor Davis, et al.

High silica cement is being used in approximately 100,000 cu. yds. of concrete going into the construction of the San Francisco-Oakland Bay bridge. The use of this type of cement is being permitted on the basis of tests started by this Department in 1932.

An extensive test series with commercial special sea-water and high silica cements started by this Department in the latter part of 1933 includes tests to determine the strength, resistance to the action of sea and concentrated sulphate waters and of volume changes compared with the standard portland cements manufactured by the same companies.

As in the case of the University of California series, the ultimate aim of the investigation is to provide information which may be used as a guide in the preparation of specifications for cements of this type and for their use in concrete construction.

The five year tests of the 1929 series are now available, as well as the three year tests of the 1931 series in both of which Fresno pumicite was used as an admixture.

The table numbers used in the 1931 report have been retained in this discussion except that the form of the tables has in some instances been modified to present the data more clearly or more accurately.

In the Imperial Valley series the pumicite was added to the standard cement content. Referring to Table 9 and Fig. 1, there appears to be a slight retrogression in strength at five years in the concrete in which pumicite admixture was used. There may be an actual retrogression in strength of this admixture pavement concrete although the evidence is not conclusive because of the relatively limited number of test specimens. A large number of cores might have produced a higher average more nearly comparable with the standard concrete without admixture.

The fact that there was no increase in strength at five years over one year is not surprising as the district in which the test was conducted is one of very low humidity and practically no rainfall. The area is approximately 120 ft. below sea level. Temperatures during the summer run to as high as 120° in the shade and the total precipitation for the five year period amounted to only 12.42 in. or an average of 2.8

TABLE 1 (REVISED)

Fresno pumicite No. 1, Monolith cement—Series started March, 1929.

The additions and substitutions were made by volume of cement to give the same yield as the standard 1:2 mix.

Mix	Compressive Strength—p.s.i.											
Substitution (by Volume)	7 Days		28 Days		90 Days		180 Days		1 Year		5 Years	
	Lb.	%	Lb.	%	Lb.	%	Lb.	%	Lb.	%	Lb.	%
1 ct.—2 sd.	4617	100	7435	100	8916	100	9416	100	9650	100.0	9741	100.0
(.9 ct. +.1 Pum.)—2 sd.	4078	88.3	6721	90.4	8111	91.0	8689	92.3	9234	95.7	9546	98.0
(.8 ct. +.2 Pum.)—2 sd.	3305	71.6	5682	76.4	7136	80.0	7987	84.8	8273	85.7	9238	94.8
(.7 ct. +.3 Pum.)—2 sd.	2610	56.5	4747	63.8	6195	69.5	7104	75.4	7604	78.8	8198	84.1
(.6 ct. +.4 Pum.)—2 sd.	1974	42.7	3805	51.2	5286	59.3	6097	64.8	6681	69.2	8410	86.3
(.5 ct. +.5 Pum.)—2 sd.	1344	29.1	2695	36.2	4052	45.4	5273	56.0	6664	69.1	7581	77.9
(.4 ct. +.6 Pum.)—2 sd.	851	18.4	1825	24.5	2955	33.1	4305	45.7	5162	53.5	6721	69.0
(.3 ct. +.7 Pum.)—2 sd.	494	10.7	1084	14.6	2175	24.4	3461	36.8	4259	44.1	4967	51.0
Admixture (By Volume)												
(1 ct. +.1 Pum.)—1.90 sd.	4727	102.4	7500	100.8	9124	102.3	9682	102.8	10409	107.9	10699	109.8
(1 ct. +.2 Pum.)—1.80 sd.	4617	100.0	7325	98.5	9215	103.3	9942	105.6	10877	112.7	11770	120.8
(1 ct. +.3 Pum.)—1.70 sd.	4331	93.8	7338	98.7	9085	101.9	9566	101.6	10391	107.7	11267	115.6
(1 ct. +.4 Pum.)—1.60 sd.	4455	96.4	7290	99.4	8890	99.7	9988	106.1	10734	111.2	11185	114.8
(1 ct. +.5 Pum.)—1.50 sd.	4396	95.2	7156	96.2	8955	100.4	10133	107.6	10890	112.9	10423	107.0

in. per year. Rain fell on but 49 days out of 1826 days. Variations in monthly and annual temperature and rainfall are shown in Fig. 2.

Because it has been said that concrete in which silica admixtures have been used will deteriorate with age in the absence of moisture, a laboratory series comparing the strength of specimens cured in water with companion specimens cured in air, entirely free from moisture after the curing period, was started in 1931, using a lean mix of four sacks of cement per cu. yd. The results of the one and three year tests in this series are now available, supplementing the data given in the October 1931 JOURNAL (Table 10).

Water cured specimens showed a steady increase in strength throughout the entire period with the 30 per cent admixture producing a decided increase in strength amounting to 159 per cent of standard at the three year period. The strength of the corresponding pumicite No. 1 admixture specimens was 163 per cent of standard at one year.

There was a decided falling off in strength of all air cured specimens including the standard mix. The reduction in strength was very little more, if any, however with the admixture specimens than with the standard concrete, the 30 per cent admixture air cured specimens showing a strength 142 per cent of standard at three years. Substituting pumicite No. 1 for up to 30 per cent by weight of the cement produced, there was an increased strength in all water cured specimens and also in the air cured specimens, except at three years where the 30 per cent substitution air cured specimens ran but 98 per cent of standard. The action of pumicite No. 2 and the silica dust paralleled

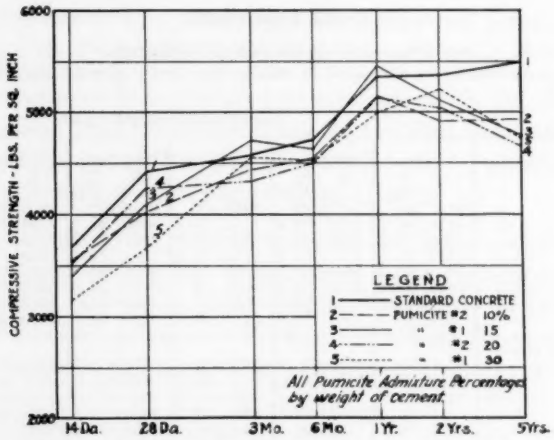

FIG. 1—COMPRESSIVE STRENGTH OF CORES FROM IMPERIAL VALLEY TEST PAVEMENT

Note—Pavement placed May 29. Tests for standard mix concrete represent from 28 to 56 cores for each breaking period; admixture concrete average of but 4 to 6 breaks for each period. In view of the fact that some of the individual cores of the pumicite concrete broke as high as 5550 lb. at five years, it is probable that had as large a number of cores been cut and tested for pumicite concrete as for the standard concrete, the results would have been more nearly comparable.

TABLE 2 (REVISED)

Fresno pumicite No. 2, Monolith cement—Series started March, 1929.

The additions and substitutions were made by volume of cement to give the same yield as the standard 1:2 mix.

Mix — Compressive Strength—p.s.i.

Substitution (by Volume)	7 Days Lb.	%	28 Days Lb.	%	90 Days Lb.	%	180 Days Lb.	%	1 Year Lb.	%	5 Years Lb.	%
1 ct.—2 sd.	4649	100	7636	100	9228	100	9812	100	10202	100.0	11056	100.0
(.9 ct. +.1 Pum.)—2 sd.	3954	85.1	6805	89.1	8533	92.5	8767	89.3	9331	91.5	10504	95.0
(.8 ct. +.2 Pum.)—2 sd.	3292	70.8	5877	77.0	7493	81.2	8267	84.3	8754	85.8	9660	87.4
(.7 ct. +.3 Pum.)—2 sd.	2519	54.2	4675	61.2	6227	67.5	7234	73.7	7681	75.3	8572	77.5
(.6 ct. +.4 Pum.)—2 sd.	1838	39.5	3565	46.7	5149	55.8	5948	60.6	6539	64.1	8198	74.1
(.5 ct. +.5 Pum.)—2 sd.	1266	27.2	2532	33.2	3922	42.5	4831	49.2	5649	55.4	7565	68.4
(.4 ct. +.6 Pum.)—2 sd.	786	16.9	1623	21.3	2727	29.6	3727	38.0	4688	45.9	6234	56.4
(.3 ct. +.7 Pum.)—2 sd.	448	9.6	941	12.3	1786	19.4	2708	27.6	3539	34.7	4026	36.4
Admixture (by Volume)												
(1 ct. +.1 Pum.)—1.90 sd.	4727	101.7	7857	102.9	9481	102.7	9850	100.4	10883	106.7	11680	105.7
(1 ct. +.2 Pum.)—1.80 sd.	4779	102.8	7662	100.3	9429	102.2	10234	104.3	11020	108.0	11998	108.4
(1 ct. +.3 Pum.)—1.70 sd.	4442	95.6	7578	99.2	9075	98.3	10066	102.6	10883	106.7	11772	106.0
(1 ct. +.4 Pum.)—1.60 sd.	4461	95.9	7117	93.2	9202	99.7	10131	103.3	11117	109.0	11884	107.5
(1 ct. +.5 Pum.)—1.50 sd.	4383	94.3	7195	94.2	9144	99.1	9728	99.2	10747	105.3	11705	105.9

TABLE 8 (REVISED)

30.0 Per cent pumicite addition (by volume of cement), Monolith cement—series started April, 1929

Yield—6 sacks cement to 1 cu. yd. of concrete.

Mix—plain concrete 1:1.80:3.66. W/C = 0.76

pumicite concrete 1 +0.30:1.62:3.66. W/C = 0.81

Slump—about 2 in. Compressive Strength—p.s.i.

	Plain Concrete		Pumicite Concrete	
28 Days	5075	100.0%	4775	94.1%
90 Days	5895	100.0%	6250	106.0%
180 Days	6375	100.0%	6860	107.6%
1 Year	6725	100.0%	6545	97.3%
3 Years	7116	100.0%	7134	100.2%
5 Years	7162	100.0%	6612	92.3%

Each result represents the average of 5-6x12 in. cylinders, except for the 5 year period. Results shown for five years represent the average of three specimens.

the action of pumicite No. 1, but the results were lower owing without doubt very largely to the coarse grading and consequent lower specific surface of these siliceous materials. Complete data covering all three admixtures are given in Table 10 (revised), and the results for pumicite No. 1 have been plotted in Fig. 3 and Fig. 4. Further test results will be available in this same series at the end of five and ten years.

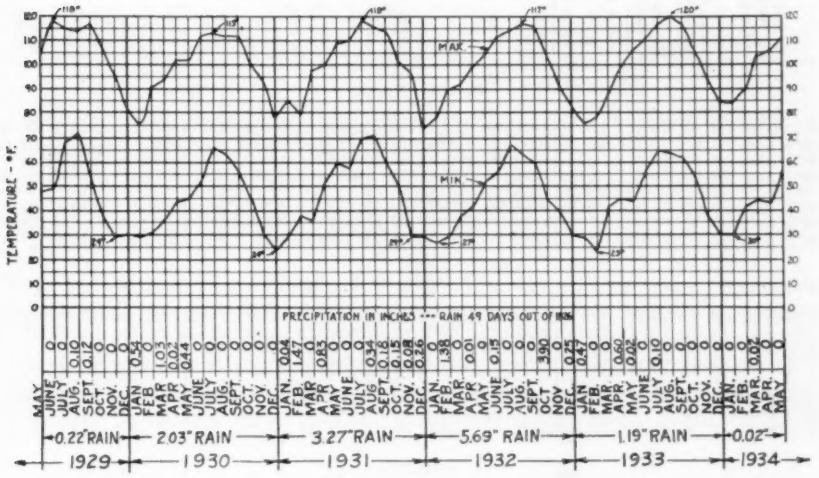

FIG. 2—TEMPERATURE AND PRECIPITATION RECORD BRAWLEY, IMPERIAL CO., CALIF., MAY 1929 TO MAY 1934

TABLE 9 (REVISED)

Imperial Valley admixture test (1929 Series)—pavement laid May 13, 1929.

All pavement dyked. Dykes kept filled with water 10 days. All admixtures were added to the standard six sack concrete.

Compressive strength of cores cut from pavement at ages indicated.

Process	Compressive Strength—p.s.i.														No. of Cores for Each Period
	14 Days		28 Days		3 Months		6 Months		1 Year		2 Years		5 Years		
	Av. Str.	% of Std.	Av. Str.	% of Std.	Av. Str.	% of Std.	Av. Str.	% of Std.	Av. Str.	% of Std.	Av. Str.	% of Std.	Av. Str.	% of Std.	
Standard 6 sack concrete	3688	100.0	4422	100.0	4593	100.0	4721	100.0	5362	100.0	5376	100.0	5498	100.0	28 to 56
Pumicite No. 1—30% (Wt. Ct.)	3170	85.9	3686	83.4	4575	99.6	4517	95.7	4994	93.1	5213	97.0	4734	86.1	4 to 6
Pumicite No. 1—15% (Wt. Ct.)	3398	92.1	4108	92.9	4721	102.8	4639	98.3	5468	102.0	5100	94.9	4764	86.7	4 to 6
Pumicite No. 2—10% (Wt. Ct.)	3563	96.6	4024	91.0	4430	96.5	4553	96.5	5161	96.3	4908	91.3	4925	89.6	4 to 6
Pumicite No. 2—20% (Wt. Ct.)	3505	95.0	4268	96.5	4322	94.1	4497	95.3	5150	96.1	5039	93.7	4656	84.7	4 to 6

TABLE 10 (REVISED)

Summary of compressive strengths four-sack admixture concrete—(1931 Series).
Standard mix—1:3.00:5.39, containing no admixture.
Specimens cast between 4-27-31 and 5-11-31.
Air cured specimens stored first summer in the open air—thereafter in a dry warm room.

% Admixture by Weight of Cement	Yield, Sacks Cem. per Yard Concrete	Water			Comp. Str.—3 Mos.				Comp. Str.—1 Year				Comp. Str.—3 Years			
					Cured in Water 3 Months		Cured in Water 28 Days–Balance in Air		Cured in Water 1 Year		Cured in Water 28 Days–Balance in Air		Cured in Water 3 Years		Cured in Water 28 Days–Balance in Air	
		Per Sack Cement, Lb.	Water-Cement Ratio	Slump in In.	Comp. Str.	Ratio to Std.	Comp. Str.	Ratio to Std.	Comp. Str.	Ratio to Std.	Comp. Str.	Ratio to Std.	Comp. Str.	Ratio to Std.	Comp. Str.	Ratio to Std.
Pumicite No. 1																
Standard	4.03	70.0	1.12	½	2408	100.0	2460	100.0	2880	100.0	1995	100.0	3310	100.0	1625	100.0
10% Add.	4.08	70.0	1.12	¾	3060	127.1	3060	124.4	3795	131.8	2620	131.4	4130	124.8	2210	136.0
20% Add.	4.07	70.0	1.12	1¼	3700	153.7	3450	140.2	4410	153.1	2970	148.9	5170	156.2	2290	141.0
30% Add.	4.05	70.0	1.12	1⅛	4010	166.6	3740	152.0	4690	162.9	3570	178.9	5270	159.2	2300	141.6
10% Subst.	3.66	77.8	1.25	¾	2640	109.7	2680	109.0	3175	110.3	2120	106.3	3930	118.8	1840	113.3
20% Subst.	3.23	87.6	1.40	½	2685	111.5	2525	102.6	3280	113.9	2155	108.0	3630	109.7	1765	108.6
30% Subst.	2.82	100.1	1.60	⅛	2293	95.2	2170	88.2	2980	103.5	1815	91.0	3610	109.1	1590	97.9
Pumicite No. 2																
Standard	4.03	70.0	1.12	½	2409	100.0	2460	100.0	2880	100.0	1995	100.0	3310	100.0	1625	100.0
10% Add.	4.03	70.0	1.12	⅛	3025	125.6	3240	131.8	3600	125.0	2585	129.6	4130	124.8	2200	135.4
20% Add.	4.04	70.0	1.12	⅛	3445	143.0	3280	133.4	4175	145.0	2900	145.4	4790	144.7	2275	140.0
30% Add.	4.03	70.0	1.12	¼	3800	157.8	3595	146.2	4375	151.9	3065	153.7	5290	159.9	2400	147.7
10% Subst.	3.65	77.8	1.25	⅛	2505	104.0	2565	104.3	2915	101.2	2015	101.0	3700	111.8	2040	125.5
20% Subst.	3.24	87.6	1.40	⅛	2350	97.6	2260	91.9	3095	107.5	1835	92.0	3485	105.3	1755	108.0
30% Subst.	2.84	100.1	1.60	½	2095	87.0	1970	80.1	2770	96.2	1575	79.0	3525	106.5	1460	89.9
Silica Dust																
Standard	4.03	70.0	1.12	½	2775	100.0	2560	100.0	2850	100.0	2040	100.0	3360	100.0	1745	100.0
10% Add.	4.05	70.0	1.12	⅛	3085	111.2	3080	120.3	3475	121.9	2345	115.0	3950	117.6	2100	120.4
20% Add.	4.07	70.0	1.12	1⅛	3375	121.6	3300	129.0	3875	136.0	2690	131.9	4400	131.0	2205	126.4
30% Add.	4.04	70.0	1.12	1½	3660	131.9	3500	136.8	4030	141.4	3090	151.5	4915	146.3	2420	138.7
10% Subst.	3.64	77.8	1.25	⅛	2670	96.2	2660	104.0	2845	99.8	2085	102.2	3560	106.0	1845	105.7
20% Subst.	3.24	87.6	1.40	⅛	2100	75.7	2180	85.2	2450	86.0	1705	83.6	3110	92.6	1650	94.6
30% Subst.	2.84	100.1	1.60	⅛	1790	64.6	1885	73.6	2235	78.4	1445	70.8	2715	80.8	1285	73.7

In the 1929 mortar series (Tables 1 and 2) in which the specimens were all cured in water at laboratory temperature for all periods, there is a steady increase in strength throughout the five years. As was expected, the percentage increase was more marked in the specimens where pumicite was substituted for a portion of the cement than with the specimens to which no pumicite was added.

In this series using pumicite No. 1 (Table 1) the standard mix specimens increased from 7435 p.s.i. at 28 days to 9741 p.s.i. at five years, or 31 per cent, whereas with the corresponding specimens in which pumicite was substituted for 70 per cent of the cement (by volume) the increase was from 1084 p.s.i. at 28 days to 4967 p.s.i. at five years or 360 per cent.

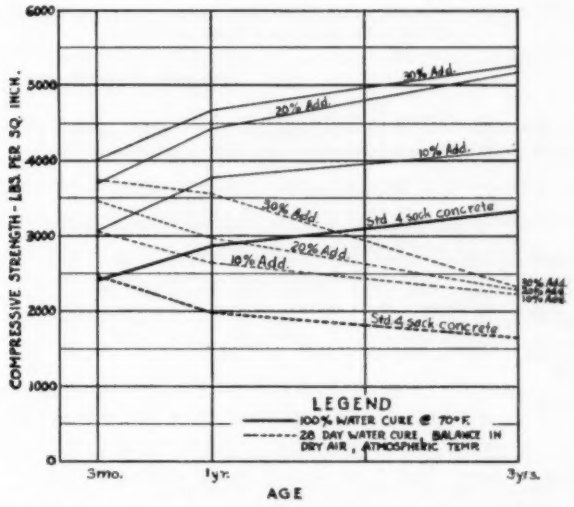

FIG. 3—COMPARING STANDARD FOUR SACK CONCRETE AND CONCRETE IN WHICH PUMICITE (NO. 1) WAS ADDED TO THE STANDARD QUANTITY OF CEMENT IN PERCENTAGES INDICATED. (SEE TABLE 10)

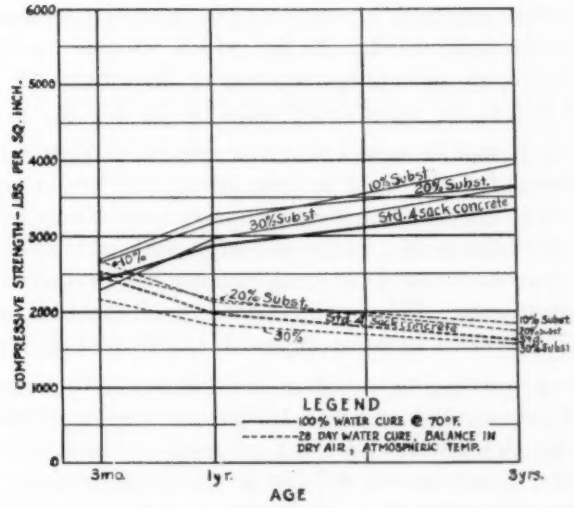

FIG. 4—COMPARING STANDARD FOUR SACK CONCRETE AND CONCRETE IN WHICH PUMICITE (NO. 1) WAS SUBSTITUTED FOR CEMENT IN PERCENTAGES INDICATED. (SEE TABLE 10)

TABLE 14—ACTIVITY OF VARIOUS SILICEOUS MATERIALS AS INDICATED BY COMPRESSIVE STRENGTH OF SILICA-LIME MORTARS

Siliceous Material	Spec. Gravity	Fineness % Passing		Compressive Sgrength—p.s.i.								
				One* Part Hydrated Lime † Two Parts Siliceous Material by Weight			Siliceous** Material			Hydrated** Lime		
		No. 200	No. 325	Water % Dry Wt.	28 Day	3† Month	Water % Dry Wt.	28 Day	3† Month	Water % Dry Wt.	28 Day	3† Month
Raw Monterey Shale	2.25	100.0	96.2	70.0	780	2930	67.5	20	52	62.5	75	71
Calc. Mont. Shale	2.39	100.0	94.8	63.3	1450	2320	64.0	45	50			
Raw Ione Clay	2.54	100.0	91.4	54.1	194	388	55.0	Soft ††	17	62.5	71	73
Calc. Ione Clay	2.69	100.0	91.3	51.6	1130	2570	51.6	62	73			
Gr. Ottawa Sand	2.65	100.0	98.9	43.3	58	168	37.1	Soft ††	16	62.5	72	71
Friant Pumicite	2.41	100.0	98.4	50.0	1370	1750	45.8	42	51			
Unground It. Pozz. (Bacoli) passing No. 200 mesh	2.42	100.0	80.6	41.3	370	980	37.5	Soft ††	Soft ††	62.5	72	73
Gr. It. Pozz. Passing No. 200 mesh	2.48	100.0	99.1	45.0	1550	2390	34.6	Soft ††	Soft ††	62.5	72	72

*Cured in moist air (approx. 100% R. H.) for 7 days and then placed in water at 70° F.
**Cured in moist air (approx. 100% R. H.) at 70° F. for entire period.
†Other test periods will be at six months—one, five and ten years.
††Specimen not sufficiently hard to register load on beam.

Similarly, with pumicite No. 2 (Table 2) the increase for the standard was from 7636 p.s.i. at 28 days to 11056 p.s.i. at five years or 45 per cent while the corresponding increase in the 70 per cent substitute specimens was from 941 p.s.i. at 28 days to 4026 p.s.i. at five years or 330 per cent.

The comparatively high strength of the standard specimens in the second pumicite series (Table 2), for which there is no explanation, served to reduce the percentage relation of the admixture specimens to standard below the percentage found in the first series (Table 1).

In Table 1, it will be noted that the specimens in which pumicite was substituted for the sand in the amount of two tenths of the cement content had an average compressive strength at five years of 120.8 per cent of standard. In the second series (Table 2) however, the ratio was reduced to 108.4 per cent because of the high test of the standard specimens. The actual breaking strength however, was 11,770 p.s.i. in Series 1 as compared with 11,998 p.s.i. in Series 2.

It will be interesting to compare the 1:2 pumicite series with the silica dust, German trass and lean 1:3 pumicite series (Tables 3, 4, 5, 6 and 7 of the October 1931 Report) as soon as the five year figures are available. These figures will not be available however, until the end of 1934 as this supplemental series was not started until Dec. 1929.

In Table 14, are recorded the short time results of a recent silica-lime mortar test now under way in connection with the test series of

special cements commenced the latter part of 1933. These tests indicate that the pozzuolanic activity of the Monterey shale is superior to that of the Fresno pumicite.

As was to be expected however, the material containing the greatest percentage of "flour" shows the greatest activity. This is particularly apparent with the Italian pozzuolana where the results of the first test using the natural product screened through a 200 mesh but with only 80 per cent of minus 325 mesh material are much inferior to the later tests using specially ground materials 100 per cent through a 200 mesh and 99 per cent through the 325 mesh. Specific surface figures are not available but will accompany later reports on this series.

*W. C. Hanna** (*Colton, Calif.*—by letter): The authors have presented an interesting paper and pointed out why the high-silica cements should not be used for most kinds of construction. They have failed, however, to emphasize to a sufficient degree where these cements are defective and dangerous for most uses.

The second paragraph on page 370 gives the chief disadvantages as "the necessity for longer moist curing . . ., and greater shrinkage in concrete subject to drying conditions." It is these two things, more than any others, which are responsible for poor concrete work. To increase either the curing period or the shrinkage certainly would not be desirable in most sections of the United States.

By a longer period of curing it is to be inferred that the moist curing should exceed 28 days since the authors state in the second paragraph of page 382 that "in dry air after 28 days (moist curing), the strength for blended cements never approaches that for the straight cements." Unfortunately very little concrete work in large sections of the country receives even as much as half the amount of curing that straight cement concrete should have for best results and if the silica cements required still more curing they certainly could not be used to advantage in most places.

Tests made by me confirm the authors' findings that greater curing time in moist condition than 28 days is needed for the silica cements. The following results of tests on 2 x 4 in. cylinders using one part of cement to three parts of standard Ottawa sand and fabricated and tested in accordance with the old A. S. T. M. Tentative Standard will be of interest.

*Chief Chemist, California Portland Cement Co.

TABLE 1

Tests with 1 cement: 3 standard sand, by weight
Specimens Stored in Water

No.		Compressive Strength (p.s.i.) 1 Da.	3 Da.	7 Da.	28 Da.	3 Mo.	1 Yr.	2 Yr.
16202	Silica cement	315	858	1717	3745	4427	4207	4646
16199	Portland cement	767	1803	2987	3890	4226	4392	4415

Part of the good showing of the "Silica Cement" was due to the greater volume of cementing material since the specific gravity of the silica cements is less than the portland cements.

The tests reported in Table 1 provide no information in regard to the behavior with standard sand under dry conditions, or in mixtures in which fine aggregate is already present, either under wet or dry conditions, after normal curing.

With the exception of some special uses, concrete is not generally submerged in water for two years, or even for 28 days. At the time that the specimens were made for the test shown in Table 1, additional 2 x 4 in. cylinders were made which were stored in water for 28 days (corresponding to good curing in actual use) and were then stored in air for the remaining period. This test is of interest because the manner of storing corresponds more closely to the conditions generally found in actual use of the cements. The results are given in Table 2.

TABLE 2

Tests with 1 cement: 3 standard sand, by weight.
28 Days in Water, then stored in air

		Compressive Strength (p.s.i.) 3 Mos.	1 Year	2 Yrs.
16202	Silica cement	4427	3929	4065
16199	Portland cement	4226	4592	4980

In this test, the portland cement shows a steady gain from 3 months to 2 years, and at 2 years the strength is 565 lb. greater than the corresponding 2 year strength with water storage shown in Table 1. The silica cement, which showed favorable results with water storage in Table 1, is lower in strength at 1 year and two years than at 3 months. Under the dry conditions, which are similar to those to which concrete is generally subjected, the admixture is evidently detrimental.

This test shows clearly the danger of assuming that tests of specimens submerged in water for the entire test period are indicative of the results which may be expected in actual use where the concrete or mortar is not submerged in water. In our tests of silica cement we

have found repeatedly that tests of specimens under dry conditions are unfavorable, while tests of specimens in a 1:3 mix with standard Ottawa sand have shown good strengths at 28 days and later periods.

The tests shown in Tables 1 and 2 show what the admixture ground with cement as a substitute for cement, will do when used with sand of uniform sand grains from which the fine particles have been completely removed by screening. Standard Ottawa sand will all pass No. 20 sieve and be retained on No. 30 sieve. To determine whether the good results under water storage were due to the lack of fine particles in the standard sand another test was made in which standard Ottawa sand was again used, but fine aggregate was supplied in the form of fine rock dust. The mortar was composed of 1 part of cement, 4 parts of standard Ottawa sand and 1 part of fine rock dust, by weight. This mortar was molded into 2 x 4 in. cylinders as in the preceding tests. The cylinders were stored in water under standard conditions. The results are given in Table 3.

TABLE 3

Tests with 1 cement: 4 standard sand: 1 fine rock dust, by weight—specimens stored in water

No.		Compressive Strength (p.s.i.)						
		1 Da.	3 Da.	7 Da.	28 Da.	3 Mo.	1 Yr.	2 Yr.
16202	Silica cement.....	236	635	1191	2245	2620	2677	2553
16199	Portland cement....	688	1933	2361	3091	3414	3596	4240

In this test the silica cement shows unfavorably in comparison with portland cement. Although the specimens made with portland cement gain steadily in strength throughout the entire 2 years, the specimens made with silica cement show very little gain from 28 days to 1 year, then retrogress, having a lower strength at 2 years than at 1 year, or even 3 months.

In this test the silica present as a fine admixture serves no useful purpose, since fine aggregate is already present. The mix is a lean mix, containing only one part of cement to five parts of aggregate, and suffers greatly from the substitution of silica for part of the portland cement. It is very clear that silica cannot replace portland cement.

At the time that mortar specimens were made for the test shown in Table 3, additional 2 x 4 cylinders were made, stored in water for 28 days, then removed and stored in air for the remainder of the test period. The results are given in Table 4.

In this test, the portland cement shows a steady gain from 3 months to 2 years, the strength at 2 years being 715 lb. greater than the corresponding 2 year strength with water storage shown in Table 3. Here

TABLE 4

Tests with 1 cement: 4 standard sand: 1 fine rock dust, by weight
28 days stored in water, then stored in air

No.		Compressive Strength (p.s.i.) 3 Mos.	1 Yr.	2 Yrs.
16202	Silica cement	2620	2876	2763
16199	Portland cement	3414	4364	4915

the silica cement shows no retrogression, but gains only 143 lb. from 3 months to 2 years, as compared with a 1500 lb. gain for the same period for portland cement.

CONVENTION DISCUSSION

Chairman Bates: Professor Davis intimated that one of the compositions, L 20, approached closely that being used in Boulder dam. In the second contract for the cement for the dam, the allowable limits on the tricalcium aluminate were raised, and of the four manufacturers now delivering cement of the low heat type, one of them consistently runs very close to seven per cent tricalcium aluminate; the other two are close to five or six per cent, and a fourth manufacturer has about the aluminate that Professor Davis indicated, three per cent, but his 4:1:1 compound is considerably lower than the 15 per cent of that L-20 that Professor Davis showed. There is a fifth manufacturer delivering cement for the dam and I have not seen the recent analyses of his product, but I think it is close to that of the first group of about five to six per cent of the tricalcium aluminate and five per cent of the 4:1:1; so there are two different kinds of cement being delivered for the job, one with about ten per cent more of the combined silicate than the other, in which there is considerable iron.

Professor Davis: On the job those four cements are being blended so that we are not getting on any one part of the job four or five per cent and on another part six or seven per cent, but are getting a blend of all those.

Chairman Bates: At this time they are blending nine or ten different cements on the dam; effective January 1, the use of the low heat cement alone was discontinued and they are blending 40 per cent ot standard portland cement with 60 per cent of low heat cement. It happens that five different manufacturers are delivering cement and I believe at times all of them are delivering two different types of cement, so that at times they are blending at the dam nine or ten different cements.

M. A. Swayze: It seems that the cement rather high in tricalcium silicate had a smaller range between the expansion in the wet cycle

and the contraction of drying and therefore in normal exposure we would have less variation the higher the tricalcium silicate.

Professor Davis: I would add something to that; these tests were started when the specimens were 28 days old—the results shown on the slide. We kept another group of specimens, companion specimens, under water for a year. We found there that the cements which were high in dicalcium silicate, while they still shrunk the most, did not shrink anything like as much as when this drying out treatment had started at the age of 28 days, and under those conditions, under that long time curing, before the drying was started, there was about the same difference between the extremes of wet and dry for the high dicalcium silicate cement as for the high tricalcium silicate cement. I think in many of our structures, thin structures such as road slabs and buildings subject to drying conditions, certainly in some parts of the country, we would want a cement which was not too high in dicalcium silicate. I am certainly not advocating such a cement as that for all purposes, but it seems like a good cement for mass concrete construction.

This discussion is concluded except for the authors' opportunity for closure in the November-December JOURNAL.

Discussion of a paper by Samuel B. Morris and Ross White:

"Vibrating Concrete at Pine Canyon Dam"*

CONVENTION DISCUSSION

Q. How were the joints cleaned on this work?

Professor Davis: The joints on this job were cleaned by brushing and, as you will notice, the platform type of vibrator was employed. I think that if the job were being started now they would not employ the wire brushes, but that was following the past practice of those who were responsible for the construction of the dam. Another thing I think the slump or consistency of the mix was not mentioned. On the average the slump was about ¾-in. I would be inclined to believe that is a little too dry unless we have very close control. My own feeling is that a 1½-in. slump (as I indicated in the paper that I read a few minutes ago[1]) under job conditions is just about right; but if you go to the very dry consistencies, there is always uncertainty, especially on very large jobs, as to just how thoroughly you are compacting that concrete. We know that with the wetter mix, 1½-in. slump, the compaction which may be obtained by vibration, may be not only very thorough but also may be obtained very quickly.

This discussion is concluded except for the authors' opportunity for closure in the November-December Journal.

*Journal Amer. Concrete Inst., Mar.-Apr. 1934; *Proceedings* Vol. 30, p. 305

[1] "Bonding of New Concrete to Old at Horizontal Construction Joints," by Raymond E. Davis and Harmer E. Davis, Journal Amer. Concrete Inst., May-June, 1934; *Proceedings Vol.* 30, p. 422.

Discussion of two papers by R. B. Young and Wilfrid Schnarr:

"Manufacturing Concrete During Cold Weather"* AND "Cold Weather Protection of Concrete"†

L. J. Rothgery,‡ (*East Lansing, Mich.*—by letter): Since concrete structures are successfully built during the coldest months of the year and winter construction work is a vital necessity if labor is to be better distributed throughout the year, the authors of the two papers on cold weather concreting have rendered a distinct service.

As a result of numerous inquiries of the Engineering Experiment Station as to satisfactory results from dry-heating aggregates, laboratory tests were made from which data are available supplementing the authors' reported experiences.

On sizable projects the better equipped contractors usually have at their disposal not only materials for enclosing the work but equipment for heating materials efficiently. There are still, however, many jobs upon which it is felt that fire heating the aggregates is more economical and just as satisfactory as steam. Several methods are employed but the usual method is to install old boiler tubes or corrugated metal pipes under the stock piles of sand and gravel and maintain fires in these pipes. Tarpaulin covers are spread over the piles to keep the heat in and as a protection against snow.

With constant draft through the pipes fairly high temperatures are reached. On a number of jobs inspected by the writer it was noted that the temperature maintained was high enough to keep the metal of the pipes in a continuous glow. Comparable conditions produced in the laboratory showed this temperature to be between 1200 and 1250° F.

An investigation was made to compare the concrete making qualities of the aggregates which were subjected to protracted periods at this temperature with aggregates in the condition of those at the edge of the pile which were only thoroughly defrosted and dried.

*Journal Amer. Concrete Inst., Mar.-Apr. 1934; *Proceedings* Vol. 30, p. 279.
†Journal Amer. Concrete Inst., Mar.-Apr. 1934; *Proceedings* Vol. 30, p. 292.
‡Field Engineer, Engineering Experiment Station, Michigan State College.

TABLE 1—TESTS OF 6 x 12 CONCRETE CYLINDERS FROM HEATED AGGREGATES

No.	Age, Days	Comp., p.s.i.	Heat	Averages	Var. from Average	% Var.
1	28	3485	1250° F.		+35	
2	"	3500	"		+50	
3	"	2365	"		—1095	
4	"	2900	"		—550	
5	"	3860	"		+310	
6	"	3815	"		+365	
7	"	3040	"		—410	
8	"	3390	"		—460	
9	"	4050*	"		+600	
10	"	4100*	"	3450	+650	31.1%
11	"	2810**	70° F. Control		—780	
12	"	3660**	"		+70	
13	"	3680**	"		+90	
14	"	3720**	"		+130	
15	"	3790**	"		+200	
16	"	3740**	"		+150	
17	"	3620**	"		+30	
18	"	3690**	"		+100	
19	"	3760**	"		—170	
20	"	3420**	"	3589	—170	5.3%

*Broken age 27 days and corrected to 28 days.
**Broken age 24 days and corrected to 28 days.

TABLE 2—TESTS OF 6 x 8 IN. CONCRETE BEAMS TESTS FROM HEATED AGGREGATES

No.	Heat	Load (W)	Modulus of Rupture	Average
1 (a)	Lab. Temp.	285.5	696	
(b)	"	251.5	623	
2 (a)	"	250.5	621	
(b)	"	251.5	623	627
1 (a)	1250° F.	240.5	598	
(b)	"	236.0	590	
2 (a)	"	255.0	630	
2 (c)	"	250.0	620	606
				Diff. 3.35 per cent

TABLE 3—ABRASION TESTS OF HEATED AGGREGATES

Sample H = Heated Aggr. C = Control	% Loss in Weight	Average	% Difference
1 Sand (H)	12.2		
2 Sand (H)	12.5		
3 Sand (H)	10.4	11.7	
1 " (C)	4.84		
2 " (C)	5.64		
3 " (C)	5.56	5.35	118
1½ " (H)	8.9		
2½ " (H)	8.5	8.7	
1½ " (C)	2.8		
2½ " (C)	2.7	2.75	222
1¾ " (H)	8.72		
2¾ " (H)	10.35	9.53	
1¾ " (C)	3.4		
2¾ " (C)	1.1	2.25	324
1–1 " (H)	5.9		
2–1 " (H)	6.7	6.3	
1–1 " (C)	1.6		
2–1 " (C)	3.1	2.35	168

Field conditions were approximated in the laboratory by constructing a gas furnace which held a charge of about 200 lb. of aggregate. Heat from simple torch type gas burners was boosted by the introduction of compressed air. A thermocouple was built into the furnace and the temperatures observed on a potentiometer type of pyrometer.

The aggregate charge was introduced into the cold furnace and the temperature raised gradually during a five hour period and held for one hour at the desired point.

The object of the investigation being to test only the effect of the extreme heat on the aggregates, the furnace and contents were allowed to cool over night before the aggregates were used in concrete. No attempt was made to use the aggregates hot and thereby introduce the effect of quenching.

The concrete was designed for 3000 p.s.i. minimum so that the mortar would have sufficient strength to develop enough bond to cause fracture of part of the coarse aggregate when specimens were tested. The aggregate mix was the same for all specimens. A consistency giving a 6-in. slump was used in all batches. Standard 6 x 12 in. compression cylinders and 6 x 8 x 36 in. beams were prepared as test specimens.

CONCLUSIONS

1. Heating these aggregates to a temperature of 1250° F. caused a breaking down on cooling of some of the softer limestone, granite and chert particles. This was noticeable in the sand as well as the coarse aggregate. The harder limestone, granite and quartzite particles showed no apparent change due to heating.

2. The difference in resistance to abrasion in heated and unheated aggregates was more noticeable than the difference in strength of concrete made from these materials. The heated sands showed a loss by abrasion 118 per cent greater than the unheated sand. The increase in this loss in the ½ in. aggregate was 222 per cent; in the ¾-in., 324 per cent and in the 1-in., the loss was 168 per cent greater. In each size of aggregate the loss by abrasion was considerably greater with the heated samples.

3. The difference in strength of concrete test specimens made from heated aggregates as compared to those made from unheated aggregates was not so noticeable. Specimens tested in flexure and in compression showed a slightly lower strength in those made from the heated aggregates. The average loss in modulus of rupture was only 3.35 per cent, and the loss in compressive strength was 3.9 per cent.

The compression specimens made from heated aggregates showed a wider range of variation from the average than was the case with the control specimens, the former showing a variation of 13.1 per cent and the latter 5.3 per cent.

4. In studying the broken test specimens, there was no apparent difference between the character of fractures of coarse aggregate particles nor the number of particles broken in specimens made from heated and unheated materials.

5. In the mixes using the heated aggregate somewhat more water was needed to give the desired consistency. It was apparent that a greater amount of water was absorbed by the heated aggregates than by the control materials to leave an amount of free water in the mixture comparable to that in the control mixes. This is true even though the aggregates used in the control mixtures had been in dry storage for a considerable time.

6. Apparently any breaking down of the aggregates due to heating occurred before or during the mixing process, this action causing a reduction in the fineness modulus rather than a disintegration of the resulting concrete. The water requirements for a given consistency varied somewhat from batch to batch which would indicate some difficulty in accurate control on jobs where aggregates are so treated. The mixtures should be designed for slightly higher strengths to offset this difficulty.

7. These results seem to indicate that production of reasonably uniform and high quality concrete is possible with this type of aggregate heating when proper precautions are taken in accordance with these conclusions. The loss in strength as indicated in the foregoing tests might easily be compensated by the use of the usual cold weather precautions and slightly stronger mixtures.

CONVENTION DISCUSSION

Convention discussion was confined chiefly to questions from the audience answered by Mr. Young, presentor of the first paper. Some of these are briefed as follows—EDITOR

Q.—What is the safe or upper limit to which the mixing water should be heated?

A.—I think there is a practical limit to the temperature at which you can handle the mixing water so that you can get it thoroughly mixed; 180 degrees is practical.

Q.—At what weather is it necessary to begin heating aggregates in addition to heating mixing water?

A.—An outside job is pretty well governed in this respect by the time when aggregates, containing frost, get lumpy. If you are running a ready-mix plant, you are probably subject to regulations of the building laws or to prejudices on the part of your customers and you have to be governed by those rather than the actual necessities of the case.

Q.—What is the effect of heating aggregates by jets of steam?

A.—We never had any difficulty. Concrete made that way will test up with concrete made under summer conditions.

This discussion is concluded except for the authors' opportunity for closure in the November-December JOURNAL.

Discussion of a Paper by William H. Thoman and Warren Raeder:

"Ultimate Strength and Modulus of Elasticity of High Strength Portland Cement Concrete"*

R. L. Bertin† (*New York, N. Y.*—by letter) The writer was particularly interested in the application by the authors of the cement-water ratio (by weight)—strength relation.

Some years ago, the writer made a study of the relation between cylinder strength of concrete and various expressions denoting the relation between the amount of cement and that of water used in the mixtures.

Fig. 1 shows the relation between water cement ratio, as defined by Abrams and other expressions considered at the time, as follows:

$$R = \frac{w}{c} \quad \text{(apparent volume)} = R$$

$$R_1 = \frac{c}{w \div c} \quad \text{(absolute volume)} = \frac{1}{1 \div 2.058R}$$

$$R_2 = \frac{c}{w} \quad \text{(absolute volume)} = \frac{.487}{R}$$

$$R_3 = \frac{c}{w} \quad \text{(apparent volume)} = \frac{1}{R}$$

$$R_4 = \frac{c}{w} \quad \text{(by weight)} = \frac{1.506}{R}$$

$$R_5 = \text{specific gravity of paste} = \frac{R \div 1.506}{R \div .487}$$

The above equivalents in terms of R are based on the following:

Sp. gr. of water = 1.0 weight of water per cu. ft. = 62.4 lb.
Sp. gr. of cement = 3.10
1 cu. ft. cement (apparent volume) = 94 lb.

This diagram was used as a means of converting the various expressions from one to any other.

Fig. 2 shows the relation between strength and the various expressions R to R_5, based on $S = \frac{14000}{7R}$.

*Journal, Amer. Concrete Inst., Jan.-Feb. 1934; *Proceedings* Vol. 30, p. 231.
†Chief Engineer, White Construction Co., Inc.

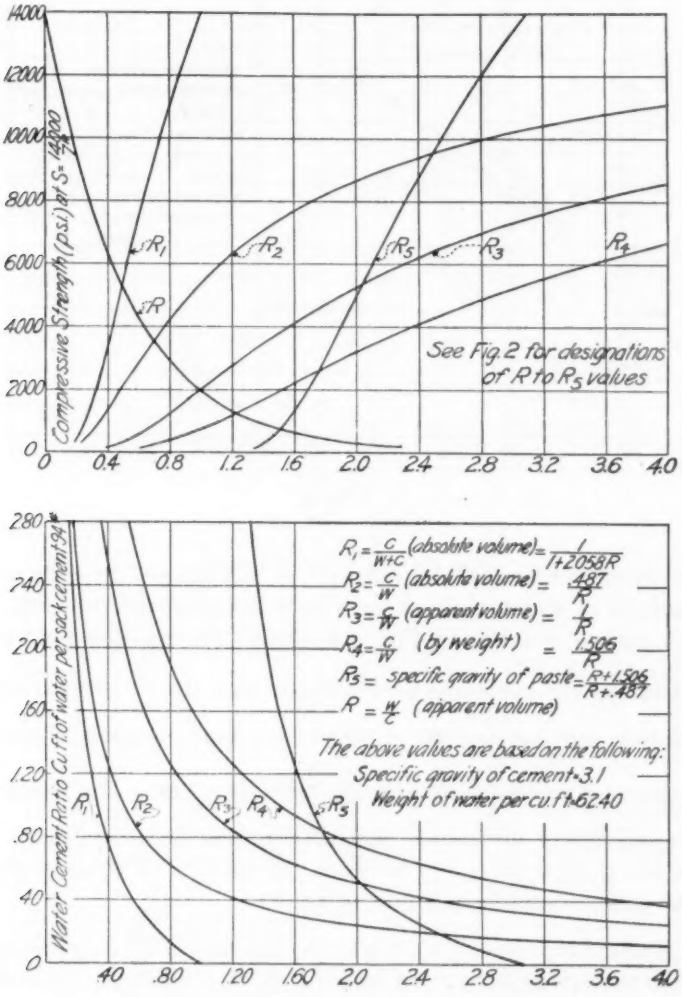
Compressive Strength (psi) at S= 14000
See Fig. 2 for designations of R to R5 values
Water Cement Ratio Cu Ft water per sack cement 94#
R1 = C/(W+C) (absolute volume) = 1/(1+2.058R)
R2 = C/W (absolute volume) = .487/R
R3 = C/W (apparent volume) = 1/R
R4 = C/W (by weight) = 1.505/R
R5 = specific gravity of paste = (R+1.505)/(R+.487)
R = W/C (apparent volume)
The above values are based on the following:
Specific gravity of cement = 3.1
Weight of water per cu ft = 62.40

FIG. 1 AND 2

The strength results of a great number of test series were plotted in terms of the various expressions R to R_5, and as a result the specific gravity of the paste was selected as the one expression with which a straight line relation approximated the test results over the greatest range of water content, partly because of its finite limits and also because it represents more logically the concentration of cement particles in the paste or in the voids of the aggregate where they are completely filled with paste.

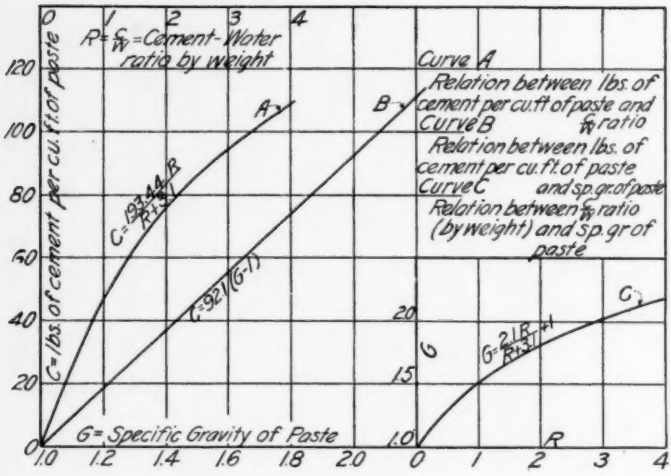
R = c/w = Cement-Water ratio by weight
Curve A
Relation between lbs. of cement per cu.ft of paste and c/w ratio
Curve B
Relation between lbs. of cement per cu.ft. of paste and sp.gr.of paste
Curve C
Relation between c/w ratio (by weight) and Sp.gr of paste
C = lbs of cement per cu. ft. of paste
G = Specific Gravity of Paste
C=321(G-1)
A
B
C
R
G

FIG. 3

The cement-water ratio by weight is in fact the weight of cement per unit weight of water and, as such it is not a direct function of the cement concentration. In Fig. 3 is shown the relation between the weight of cement per cubic foot of paste and the $\frac{c}{w}$ ratio (curve A), the relation between the weight of cement and the specific gravity of paste (curve B), and the relation between the $\frac{c}{w}$ ratio (by weight) and specific gravity of the paste. An examination of these graphs shows clearly the radical difference between the relation of these expressions and the cement content per unit of volume.

Fig. 4 and 5 represent the data of the tests conducted by the authors in which the strength to paste specific gravity relation is shown, also that of the strength to cement water ratio, the latter being a reproduction of Fig. 1 in the original paper.

A study of these diagrams shows that the last points, *RG* of Fig. 4 and *WG* of Fig. 5, plotted against specific gravity do not have the deviation from the average straight line curves that exist when plotted against the $\frac{c}{w}$ ratio by weight.

When plotted against specific gravity of paste, the general alignment of the points warrants the inclusion of all the points in passing a

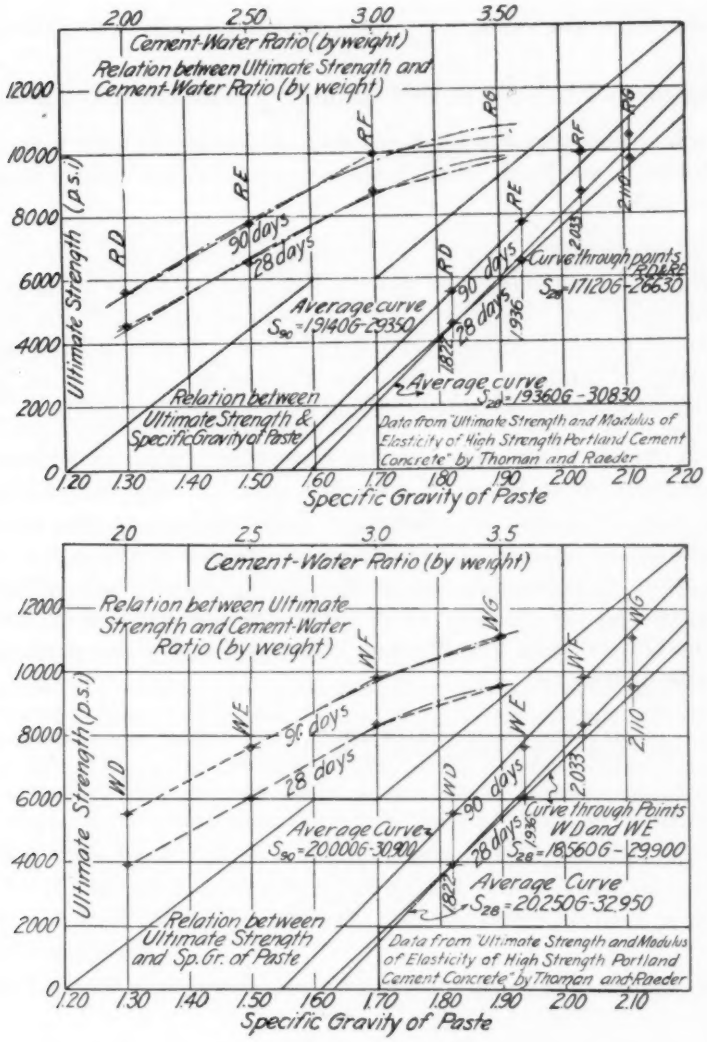
Cement-Water Ratio (by weight)
Relation between Ultimate Strength and Cement-Water Ratio (by weight)
Ultimate Strength (p.s.i.)
Specific Gravity of Paste
Average curve
Relation between Ultimate Strength & Specific Gravity of Paste
Data from "Ultimate Strength and Modulus of Elasticity of High Strength Portland Cement Concrete" by Thomas and Raeder
Curve through points
Cement-Water Ratio (by weight)
Relation between Ultimate Strength and Cement-Water Ratio (by weight)
Ultimate Strength (p.s.i.)
Specific Gravity of Paste
Average Curve
Relation between Ultimate Strength and Sp. Gr. of Paste
Curve through Points WD and WE
Average Curve
Data from "Ultimate Strength and Modulus of Elasticity of High Strength Portland Cement Concrete" by Thomas and Raeder

FIG. 4 AND 5

curve through any of the sets of data. The fact that these curves are linear, leads to the conclusion that the increase in strength is directly proportional to the increase in the specific gravity of the paste, or its equivalent, the increase in cement per unit of paste, for the full range of the tests.

Referring to the strength $\frac{c}{w}$ ratio relation, it follows that the curves fitting the points when plotted against $\frac{c}{w}$ ratio should also include all the points and that the relation for the full range of tests is therefore not linear, a condition which is to be expected in the light of diagrams shown in Fig. 3.

The proportional limit of the strength—specific gravity relation is reached when the amount of water is less than the voids in the cement at the upper end of the curve; and at the lower end, when the amount of water taken into consideration is in excess of that which the cement is capable of retaining, in which case the specific gravity is no longer a gauge of the uniform concentration of cement in the paste.

It is outside of the scope of this discussion to deal with the many factors, such as inclusion of materials, inert or otherwise, as fine as cement in the paste, the relation of paste stiffness to size of voids, segregation of fluid pastes, shrinkage of the paste within the voids, absorption of aggregate, and many others, all of which affect the strengths and the trend of their relation to any expression denoting the relation between the amount of cement and water used in the concrete mixtures.

This discussion is concluded except for the authors' opportunity for closure in the November-December JOURNAL.

Discussion of a paper by H. N. Walsh:

"Simplified Concrete Mix Design"*

F. N. Menefee† (*Ann Arbor, Mich.*—by letter). The author has presented a graphical method for determining gradation of aggregate for producing dense, workable, strong concrete. Inasmuch as the presentation refers to the work of European engineers in developing grading curves in recent years and states that their researches form the basis for the method shown, the writer is taking the liberty of directing attention to part of what has been done by the American engineer in the same line, that readers may have the benefit of both methods.

Fuller and Thompson[1] made extensive researches on gradings of concrete aggregates mixed with and without cement, drawing many conclusions which are still considered sound and in complete agreement with the author. For instance, in referring to matrix with a large proportion of the largest size stone, Fuller says:

> Trial mixes of concrete made in the laboratory with uniform coarse stone gave greater density than could be obtained with uniformly graded stone. However, in the former, nearly 50 per cent of the weight of the total aggregate consisted of particles of the coarsest diameter of stone and in mixing... it was considered doubtful whether the concrete would *work* well in practice because of the apparent tendency of the mortar to run away from the stones, and leave pockets of stone and voids.

Fuller then points out that the uniformly graded material, although somewhat less dense, gave greater strengths, and assigned as a reason lesser homogeneity in the matrix where the large stone was of one particular grading.

The author's experience seems to corroborate Fuller in this, for he says:

> For a fixed maximum particle size, one particular grading will give a greater weight per cu. ft. than any other, but the mixture so graded to give a maximum density will be harsh and difficult to place, especially if the biggest particles in it are more than one inch in diameter.

From Fuller's 21 conclusions, one may sum up the pertinent matter regarding density, strength, permeability, and workability of concrete,

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*Journal, Amer. Concrete Inst., Nov.-Dec. 1933; *Proceedings* Vol. 30, p. 110.
†Professor, Dept. Engineering Mechanics, Univ. of Michigan
[1]*Proc. A. S. C. E.*, Vol. 33, 1907, part I, p. 223.

by saying that if the mechanical analysis of the aggregate forms a parabola, whose equation is $P^2 = \frac{10{,}000}{D}d$

where P = percentage aggregate passing any given diameter or sieve opening d. (Ordinate on Fuller curve)

d = size of sieve opening (abscissa on Fuller curve),

D = size of largest particle,

that gradation of this particular material will, when mixed with a given amount of cement, be densest consistent with workability and give the strongest concrete, provided, of course, other controlling influences are comparable. In this he differed from Feret, who for mortar at least advocated no medium sized sand grains.

Such a curve is shown in Fig. A. By adding the cement to the aggregate that portion of the curve below 1/10 size of largest particle is raised to the percentage represented by the combined weight of sand and cement. Fuller advocated a modification of the parabola for cement, sand and gravel to an ellipse starting at 0 and running to an abscissa of 1/10 size largest particle and ordinate 37½ per cent passing, and a straight line from that point to size of largest particle, 100 per cent passing.[2] Some advocate 40 be substituted for 37½. This would mean adding more cement or more fines. By reference to Fig. A, it will be noted that the parabola crosses the 1/10 size largest particle at about 31.6 passing, whereas Fuller's suggested ideal curve for cement and aggregate crosses at 37½ or 40 per cent. The dotted line in Fig. A is a plot of the author's 1:4 mixture shown in his Fig. 3. It will be noted that it represents a material with slightly less fines than Fuller's original parabola. Likewise, that it represents a material with about the same amount of fines as are indicated by the dashed line curve which is a plot of sample 6, Table No. 2 in "Concrete Data for Engineers and Architects," Portland Cement Association. Sample No. 6 had a fineness modulus (Abrams) of 5.75 and a strength of 3540 p.s.i. From Table 2, and Table 3 following it, it appears that out of 19 other mixes, those with fineness modulae lesser or greater than 5.75 all gave weaker concretes. In general, this corroborates the experience of the writer and seems to check with Fuller. For with the cement added, curve for sample 6 would rise to 40 per cent or more passing at sieve .15, thus approximating Fuller's ideal for cement and aggregate for that point, although it would be higher from there on. What has been said of sample No. 6 curve is applicable to Fuller's parabola and the author's 1:4 mix as far as their elevation at 1/10

[2]Sometimes called Fuller's Ideal.

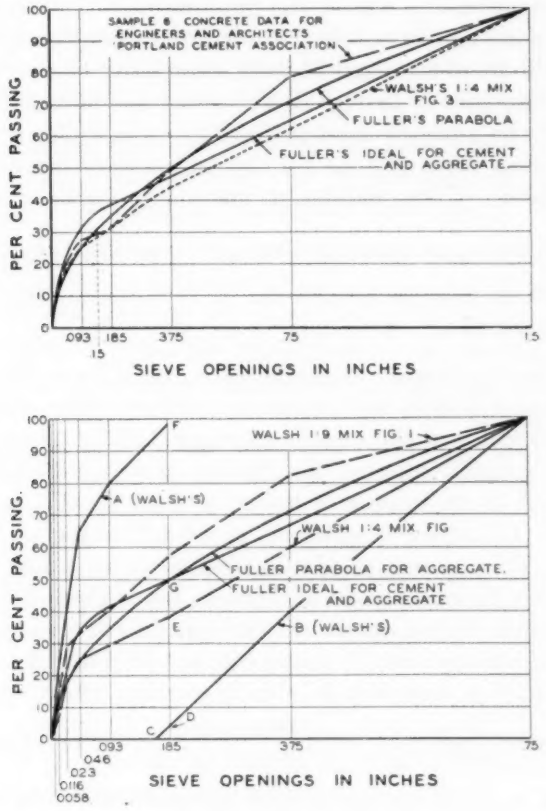
SAMPLE 6 CONCRETE DATA FOR ENGINEERS AND ARCHITECTS PORTLAND CEMENT ASSOCIATION
WALSH'S 1:4 MIX FIG. 3
FULLER'S PARABOLA
FULLER'S IDEAL FOR CEMENT AND AGGREGATE
PER CENT PASSING
SIEVE OPENINGS IN INCHES
WALSH 1:9 MIX FIG. 1
A (WALSH'S)
WALSH 1:4 MIX FIG
FULLER PARABOLA FOR AGGREGATE.
FULLER IDEAL FOR CEMENT AND AGGREGATE
B (WALSH'S)
PER CENT PASSING.
SIEVE OPENINGS IN INCHES

FIG. A AND B

size of largest particle is concerned. Particular attention is invited to Fuller's parabola and curve of sample 6, as an indication of the thoroughness and correctness of Fuller's work, for they each have practically the same fineness modulus; the parabola being the closest continuous curve to the broken curve giving the densest aggregate. The author's 1:5 and 1:6 mixes fall very close to Fuller's ideal curve and will have a better fineness modulae. Tables 2 and 3, above referred to, include 19 other gradations having lower fineness modulae and plotting above the Fuller ideal, and higher fineness modulae plotting below the Fuller ideal; all gave strengths less than 3540 p.s.i. Hence, it is to be inferred that Abram's fineness modulus method where it plots somewhere near Fuller's parabola corroborates the Fuller graphical method.

In his discussion of Table 1, the author states that the proper proportions of sand and gravel for his 1:4 mix in his Fig. 1 should be in the ratio of 36 lb. of sand and 64 lb. of gravel, but he does not say how he obtains that ratio. In Fig. B, the writer has plotted the values of A and B of the author's sand and gravel on a Fuller type of chart. The Fuller ideal curve and the 1:4 and 1:9 curve of the author's Fig. 1 are also shown. Using the Fuller method to combine A and B to give the author's 1:4 curve, proceed as follows. Take $\frac{DE}{DF}$ or $\frac{34}{94} =$ 36.2 per cent of fines, or of material A. This leaves 63.8 per cent of coarse, or material B.

Considering the sample of combined material to be 10 lb., 3.62 lb. will be fine and 6.38 lb. will be coarse. Ninety-eight per cent of the fine material will pass the .185 in. sieve, or 3.54 lb. Four per cent of the coarse material will pass the .185 in. sieve, or .254 lb. Thus of the combined 10 lb. sample of material, 3.54 + .254, or 3.794 lb., will pass the .185 in. sieve, which is the point E on the author's desired 1:4 curve. This process of plotting gradation of sands and gravels has been in use for many years and checks the author's figures exactly. By its use, any desired previously determined curve of materials can be obtained from any given fine and coarse materials. To combine the author's A and B materials to give a mixture which will plot very close to Fuller's parabola, take $\frac{DG}{DF} = \frac{46,}{94}$ or 49 per cent, of fines. In this case, a 10 lb. sample of the combination would have 4.9 lb. of fines and 5.1 lb. of coarse. Ninety-eight per cent of the 4.9 lb. will pass the .185 in. sieve, or 4.8 lb. Four per cent of the 5.1 lb. of coarse material, or 2 lb., will pass the .185 in. sieve. These two added together make 5 lb., or 50 per cent of the combined material which will pass the .185 sieve, and likewise is point G on Fuller's ideal curve. The writer believes that the combination of A and B materials which will follow the Fuller parabola will give a more workable mix and a stronger concrete than where the combination is made to follow the author's curve. By adding the ordinates above the author's curve and above Fuller's parabola, we obtain a fineness modulus for the author's recommended gradation of 610 and of the Fuller gradation of 573. At any rate, for a given amount of cement, it will be more workable, so it would seem that the fineness modulus (610) of the author's 1:4 mix, Fig. 1, is too great for the best possible strength and workability, or by comparing his curve with the Fuller parabola, it seems to fall too low on the chart, which is just another way of saying that it is

coarser than should be deliberately designed. According to "Concrete Data for Engineers and Architects," this mixture would fall below 3000 p.s.i., whereas with more fines or middle sizes to lift it on the chart, it would approach a possible 3500 p.s.i. in 28 days.

The author states that his "type curves" do not involve the use of artificial gradings. Neither does the Fuller parabola, although it may be used to make artificial gradings if one wishes.

It appears that the author uses a No. 4 sieve as the dividing line between fine and coarse material. This is as good a division point as any, although the tendency in United States is toward calling anything sand which will pass the ¼ in. screen, provided there are fines in it enough to fulfill density requirements. Michigan State Highway specifications require that 100 per cent of material submitted as sand shall pass a ⅜-in. circular hole sieve and that 90 per cent shall pass a ¼-in. sieve of the same type. Fuller made no such division, although his experiments were based on sands of various origins, most of which show maximum size to be about .2 in. He did place the break in his ideal curve at .1, the size of the largest particle, and in making combinations of fine and coarse material, many engineers combine so that the analysis curve of the combination passes through 00; 1/10 size largest particle, 37½ passing, and largest size particle, 100 per cent passing. However, it is best to make the combination so that the gradation curve of the combined material will pass through the intersection of the ideal curve and the abscissa representing the dividing line between fine and coarse material.

The writer has had the author's type curves for ¾ in. material checked, and finds that the upper curves in his figures give the densest mixtures, and that his lower curves can be brought up to the highest curve in each case by adding fines of proper proportion to the whole sample. For instance his 1:9 and 1:4 curves in Fig. 1 are shown herewith (Fig. B) on the Fuller type of chart. Assume anything below .093 to be sand and that to the material represented by this curve we are going to add 1/9 more sand. (This is the amount of cement that is added.) If the original sample was 9 lb., 40 per cent of it, or 3.6 lb., passed the .093 sieve. All of the new fine material, or 1 lb., will pass, so of the combination, 4.6 lb. out of 10 will pass, or 46 per cent. Thus the curve designated as 1:9 would be raised to 46 per cent at .093 in. size.

Now treat the 1:4 curve in the same way. Say we have 4 lb. in the sample of aggregate, 30 per cent of which passes the .093 in. sieve.

Thirty per cent of 4 lb. = 1.2 lb. Now add 1 lb. of sand finer than .093 and we have 2.2 lb. of the combination that will pass the .093 in. sieve. Two and two-tenths lb. out of 5 = 44 per cent, or the lowest curve is raised to practically the same elevation as the upper when the upper material has 1/9 of its weight in fines added and the lower has ¼ of its weight in fines added. They will have the same fineness modulus and will both have the same density, and they will lie slightly above Fuller's ideal for cement and aggregate.

Reference to Fig. B, upon which the author's type curves 1:4 and 1:9 from Fig. 1 are plotted, shows the Fuller parabola falling approximately one half way between them, and that by the addition of 10 per cent fines (sand or cement) to the material represented by his 1:4 type curve, it will be raised to almost exactly correspond to Fuller's ideal.

It is interesting to note that the matter of workability was recognized by Fuller who after determining the curves giving the greatest density and strength said:

> It was found by experiment, that more sand was required in the curves used in the beam mixtures than was indicated by the best ideal curves in the volumetric tests, for density . . . the first mixtures were made with curves corresponding to the best curves of the density tests. It was found necessary to raise them about 2 per cent at the tangent point to prevent a very rough surface on the beam.

In other words, as practically all invesitgators have found, there is a direct relationship between density and strength, but that for practical reasons each must be sacrificed slightly to obtain the desired workability.

In conclusion:

1. Fuller's curves and those of the author are based on the same fundamental principles.

2. Abram's fineness modulus reduced the rate of acceptance of Fuller's curves, but was based on the same fundamental principles.

3. Although Fuller's curves are not universally used by American engineers and architects in designing mixtures, many do use them and get the same results as are shown by the author.

*Inge Lyse** (*Bethlehem, Pa.*—by letter). Professor Walsh presents a very interesting account of recent European development in the art of design of concrete mixes. The methods presented in his paper are similar to those of mechanical analysis so prominent in this country

*Research Associate Professor of Engineering Materials Lehigh University, Bethlehem, Pa.

10 or 15 years ago, including the fineness modulus and the surface area methods. This country has passed through the same cycle of thought as that presented by Professor Walsh, has modified and advanced its methods of design, and definitely turned away from mechanical analysis in favor of the trial method, except in special cases such as in concrete product plants where artificial gradings may be economically justified. No single achievement has contributed so much to this advancement as has F. R. McMillan's new philosophy of concrete mixtures presented in his book: "Basic Principles of Concrete Making." This explains simply all the intricacies of concrete mixtures and the answer to questions such as the one on relationship between high strength and high density of concrete of a given cement content becomes so obvious as to require no study. Realizing that the density is the total volume less the water content (or total voids when the amount of air voids are appreciable), the increase in density is nothing but the decrease in water content and consequently a higher concentration of cement particles in the paste, with a correspondingly higher strength.

The problem of designing concrete mixes is not one of finding the ideal gradation of aggregates or the ideal concrete mix, but primarily one of economy. The desired quality of the concrete may be obtained in different ways by different concrete mixes. That mix which produces the given quality of the hardened concrete at the least cost is the most desirable mix, regardless of type or gradation of aggregates, regardless of type and brand of cement, regardless of workability of the concrete and method of curing. The gradation problem may very easily be overemphasized as is illustrated in Fig. 1* which shows that the economy of a concrete mix is only slightly affected by the gradation of the aggregates. The workability of concrete is a very elusive term and the most workable concrete mix is the one which produces the desired quality of the hardened structure at the least cost. Vibrated concrete has therefore entirely different workability qualities than hand placed concrete. An economic study of the trial batches combined with the study of placing seems therefore to be the only method of design suited for our present conception of concrete making. It is also the simplest one of all methods. The ordinary trial method may be further simplified by a method recently developed at the Fritz Engineering Laboratory and described in detail in Research Circular No. 89 of Lehigh University entitled "Simplifying Design and Control of Concrete."

*This is Fig. 8 in "Relation Between Quality and Economy of Concrete," JOURNAL, Amer. Concrete Inst., Mar.-Apr., 1933, *Proceedings* Vol. 29, p. 325

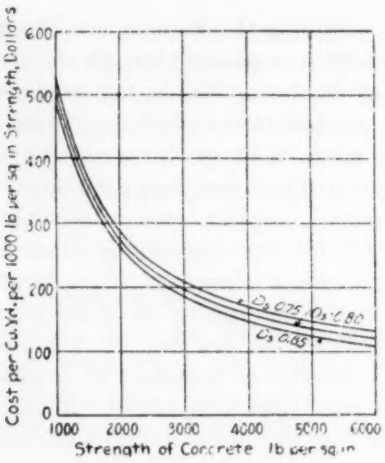

FIG. 1

Professor Walsh presents the strength formula for concrete proposed by Bolomey:

$$S = K\,(c/w - 0.5)$$

This formula is much similar to the one proposed by the writer:

$$S = A + B\,c/w,$$

except that no such general constant as the 0.5 is incorporated in the latter formula. As early as 1925 the Swiss Committee on *Gussbeton* in its report, part 9, by E. Stadelmann, proposed the strength formula

$$S = a\,\frac{c}{w} - b,$$

which, except for the sign of the constants, is identical with the writer's formula. Bolomey's formula gives a strength of zero for $c/w = 0.5$ or zero strength for a water-cement ratio by volume of about 3.0. Professor Abrams' original curve in Bulletin 1 of the Portland Cement Association, as well as a number of other results from the Association's laboratory, have shown that the concrete has a certain strength with considerably greater water-cement ratio than 3.0. Consequently Bolomey's formula does not represent the strength relationship for very lean mixes. Experiments with very rich mixes have also shown that above a certain limit the strength ceases to be directly proportional to the variation in the cement-water ratio. Furthermore, tests with different cements have shown that the strength curves do not intersect at a given point, which they should do according to Bolomey's formula. The Bolomey formula is therefore considerably more limited than the formula proposed by the writer, although as has been pointed out on several occasions, even the writer's formula

does not apply for extreme cases. It seems that the important point to keep in mind is the fact that the strength of the concrete is controlled by the concentration of cement particles in its paste and that for ordinary mixes the strength increases in direct proportion to the increase in cement concentration.

Professor Walsh has made a valuable contribution by revealing to the Institute the method of design of concrete mixes considered the best in European practice.

This discussion is concluded except for the author's opportunity for closure in the November-December JOURNAL.

Discussion of a paper by F. R. McMillan and T. C. Powers:

"A Method of Evaluating Admixtures"*

Chairman, P. H. Bates (at the convention in introducing Mr. MacMillan for the presentation of the paper): I have always been curious as to what was the first admixture. I have come to the conclusion that, without much doubt, the first admixture was portland cement. Before or as portland cements were appearing on the market, we had a series of "improved cements." For instance, the famous "Chinese wall," in Philadelphia (the structure under which we used to come into Broad street Station) was built with what was known as "improved hydraulic cement." It consisted entirely of natural cement to which portland cement was added. Possibly, in due time, the improvement reached the extent of one hundred per cent portland cement. Now we have the condition being reversed; certain types of a cement are appearing on the market as, you might say, improved portland cements, the various masonry cements improved by this, that and the other thing. I wonder what would have happened if the first individual who thought of suggesting an admixture for concrete, had used a little sales psychology and said "Since the public considers concrete as being composed only of cement, water, fine and coarse aggregates, I will antagonize nobody and instead of selling an admixture, I will simply say 'I am selling a fine aggregate'." I wonder whether a lot of discussion would not have been avoided?

George A. Smith† (*Washington, D. C.*—by letter) The paper by Messrs. McMillan and Powers, published in the March-April Journal, in which they have shown a method of evaluating various admixtures, particularly from the standpoint of their ability to promote workability in concrete mixtures, merits careful study.

Although the writer now has no personal interest in any admixture, he was, until recently, actively engaged for a number of years in studying the question of workability and the evaluation of admixtures. The results presented by Messrs. McMillan and Powers parallel those of the writer in many respects and the data they have presented in most instances check with those he has obtained.

*Journal, Amer. Concrete Inst., Mar.-Apr., 1934; *Proceedings* Vol. 30, p. 325.
†Sec'y, Code Authority for Ready-Mixed Concrete Industry.

The authors say:*

> Any of the powdered materials tested, or blends of two materials, when used in such proportions as to produce a paste of given volume and consistency, gave when combined with the same kind and grading of aggregates, concrete mixes of substantially equal workability.

Data obtained by the writer show that this is generally true in the cases of the granular and crystaline types of admixture, such as Barnsdall, talc, pumicite, etc. However, in reference to high grade diatomaceous silica the statement is not consistent with the writer's observations. Nor is it consistent with data obtained with different brands of cement which have shown widely different degrees of workability when tested under comparable conditions as to water content, consistency (flow), and aggregate proportions and gradation. Although the plastic characteristics of most admixtures play little if any part in improving workability, their principal effect being that of increasing the paste volume, the writer is convinced that diatomaceous silica does impart to a concrete mix plastic characteristics beneficial to the workability.

Table 1 shows data obtained with a remolding apparatus, similar to that used by the authors, in a series of tests conducted in the Johns-Manville Laboratories.

TABLE 1—COMPARISON OF REMOLDING EFFORT WITH DIFFERENT ADMIXTURES [1]

Grams of Water Used	Admixture			Flow	% Paste by Volume	Remolding Effort, ½" Jolts
	Kind	Lb. per Sack	Weight (Grams)			
1217	Cement	31.5	588	92	30.2	77
1133	None	0	0	90	27.2	108
1258	Celite	3	56	91	29.0	73
1275	Lime	15	282	92	30.4	77
1170	Barnsdall	5	94	89	28.1	103
1162	Pumicite	5	94	91	28.0	97
1175	Loomite	5	94	89	28.1	114
1170	Colloy	5	94	90	28.1	105

Mix—1:2:4 by dry rodded volume.
Sand and gravel proportions, and gradation constant.
Consistency constant.
Basic cement content 1765 grams per batch

It will be noted that with cement as the admixture the paste volume was 30.2 per cent, while the mix to which Celite was added had a paste volume of only 29.0 per cent. In spite of this difference the workability as indicated by the remolding effort was practically the same. In the case of lime and additional cement, the paste volumes were practically identical for the same workability. With the other admixtures no pronounced improvement was noted, probably due

*P. 329.
[1] *Proceedings*, Amer. Concrete Inst., Vol. 28, 1932, p. 693.

to the fact that they had little effect on the workability and that only relatively small quantities of each were used.

In 1927 a group of tests was made with two cements of the same brand; one an ordinary portland, the other a plastic cement. The tests were made in the laboratories of the Celite company, using the Improved Penetration Apparatus.[2] The results are given in Table 2.

TABLE 2—COMPARISON OF CEMENTS USING PENETRATION APPARATUS

	W/C	Flow	Relative Workability
Plain Cement	.990	100	100
Plain Cement, Celite 3% by Wt.	1.100	95	218
Plastic Cement	1.039	95	297
Plastic Cement, Celite 3% by Wt.	1.147	97	427

In these tests the plastic cement showed up decidedly better than the plain portland cement plus Celite, even though the paste volume was less. Hence it appears that the nature of the cement, as well as the paste volume, played some part in developing the workability of the concrete.

That the law of paste volume, if we may call it that, does not hold absolutely is indicated by the authors on Page 333, where they say, "The greatest disagreement was found with diatomaceous earth. With 27.8 per cent paste and the computed percentage of admixture, the remolding effort was less than 35; that is, the mix was too wet." The fact that the remolding effort was less than that desired is not necessarily an indication of over-wetness. Had a less volume of paste been used than the 27.8 per cent, reducing both admixture and water instead of stiffening the mix by reducing the water only, a different ratio indicating the merit of this material as compared with cement would have resulted.

In the author's Table 3 (p. 339) the effectiveness of diatomaceous silica and lime are given as 5.00 and 1.76 respectively, when compared with cement from the standpoint of promoting workability. In the writers' Table 1 presented here, the ratio for diatomaceous silica is 10.4, and for lime is 2.09. There is, therefore, a marked disagreement between the above ratio of 10.4 and the 5.0, obtained by the authors, with the diatomaceous admixture, but the values for hydrated lime show fair agreement.

In the work the writer has done with the Improved Penetration Apparatus, the ratio of the effectiveness of Celite to cement in promoting workability has consistently been determined to be between

[2] *Proceedings, A. S. T. M.*, Vol. 28, Part II, 1928.

8 and 11. With less plastic cements the ratio is high and with cements that produce a higher degree of workability, the ratio is low. His work has shown that both types of apparatus,—the Penetration Test and the Remolding Test,—show similar trends under like variations in mix compositions and many determinations of this ratio with the Penetration Test confirm the ratio of 10, indicated above as being more nearly correct than the 5 ratio obtained by the authors. In fact, the ratio indicated by the data in their Table 2 (p. 333) for that quantity of diatomaceous silica, approximately equal to that recommended ordinarily by the manufacturer, (3.5 per cent), is 7.0.

McMillan and Powers report the use of admixtures as replacements for cement. More properly, admixtures are added to a mix to improve the workability. This has been the writer's procedure, but it really makes very little difference how the matter is handled. The same result is attained in the end if all factors are given proper consideration. The authors have striven to accomplish a given degree of workability by adding large quantities of admixture. The writer has adhered to the use of quantities approximating those recommended by the producers. With those materials only little better than cement itself and using only relatively small quantities per sack of cement definite and satisfactory indications were difficult to obtain. It is the writer's opinion that using large quantities of the granular and crystalline types of materials, as the authors have done, is the better procedure and that more positive and definite ratios are possible. Notwithstanding the use of smaller quantities of admixtures, the writer's determinations of similar ratios for these materials have given values of the same order of magnitude as those indicated in Table 3 (p. 339).

It is pointed out (p. 336) that the ratio of 1.56 for Barnsdall is apparently constant, due to the fact that the lines indicating the functioning of the material in promoting workability are straight and parallel. Whereas, the authors have maintained a given degree of workability by adding the quantity of admixture necessary to accomplish the condition, the writer has noted the improvement when small amounts have been used and the mixed gaged to the same consistency. The improvement in workability has almost invariably shown a steady increase as the quantity of the admixture was increased. A specific group of tests indicating this was made, using different percentages of Celite and Talc. The results shown in Table 3 indicate an almost straight line relation between improvement in workability and the quantity of admixture used.

TABLE 3—COMPARISONS OF TALC AND CELITE USING THE PENETRATION APPARATUS

Admixture Lbs./Sack of Cement	% Improvement in Workablity	
	Talc	Celite
1	3	28
2	6	50
3	9	74
5	16	113
10	34	

It has been observed that this straight line effectiveness ratio for admixtures also holds for cements when they are used in addition to that specified for a given mix, and when the concretes are gaged to the same consistency. Reference to Fig. 6 of the writer's discussion (Vol. 28, p. 697) of Mr. Powers earlier paper[3], shows that this condition was indicated both by the Remolding Test and the Penetration Test.

The authors have shown that variations in the consistency of a mix may produce large changes in the molding effort or the workability. It is readily seen that if the plain mix with which mixes containing admixtures are to be compared, had an average slump of about 5 inches and the mixes in which admixtures were used had an average slump of about 3 inches, the result would be less favorable for the admixture, than if all mixes were gaged to have the same slump of 5 inches. For this reason the writer believes that the practice of using an admixture as an admixture, and even using much larger quantities in the case of the less effective materials than he has used and are recommended by the manufacturers, would be preferable to the condition of equal paste volumes used by the authors. This procedure lends itself readily to maintaining a close control on the consistency, regardless of the type of material under consideration and the results can be studied from the standpoint of equal paste volumes. Straight lines representing improvement will indicate effectiveness ratios in an equally good manner. Furthermore, it is believed, deviations from the law of equal paste volumes, resulting from the characteristics of the materials used, will be brought out much more effectively.

The authors, in addition to comparing admixtures with cement for workability also compare them with cement from the standpoint of their ability to promote strength. In concrete, portland cement is primarily the source of strength, and it is only an exceptional material among powdered admixtures that has the ability to produce strength pound for pound with cement. In the Table No. 3 on Page 339, the

[3] "Studies of Workability of Concrete," by T. C. Powers, JOURNAL Amer. Concrete Inst., Feb. 1932; *Proceedings* Vol. 28, p. 419.

strength indices shown are not surprising when it is remembered that few materials compare with cement for promoting strength. The fact that the indices for the 400 pounds of cement per cubic yard are greater than in the 500 pound mix, is probably an effect brought about by the improvement in the workability of the leaner mix. The results indicate that, in general, powdered admixtures have little effect on the strength. However, where adequate strength will develop from the use of a given quantity of cement, and an improvement in workability is desirable, the economy of any admixture is a function of its workability index and cost, as compared with cement.

Although the writer believes that the test method used by the authors can be improved upon and has expressed himself to that effect, and although he has taken exception to certain parts of the paper presented, his experience, and study of the subject, dictate a general acceptance of the data and concurrence in the findings presented so far as they relate to the value of powdered admixtures for promoting workability. In this view three definite points are brought out by the paper:

1. Admixtures are more or less effective in promoting workability depending upon their physical and chemical characteristics. MacMillan and Powers have shown that from the standpoint of promoting workability, the difference can be measured.

2. Cement in concrete is generally recognized as a prime factor in providing workability, even though cements may vary in their ability to do this. Comparing admixtures with a basic ingredient of the concrete is therefore a logical method of evaluating admixtures.

3. With indices of the performance of materials available, anyone considering the use of an admixture can be guided in his choice of material to give the result desired in the most economical way. Furthermore, by the use of the indices, specifications can be written, taking into account the values of the materials and putting them on a performance basis.

This discussion is concluded except for the authors' opportunity for closure in the November-December JOURNAL.

Discussion of a paper by E. Probst:

"Plastic Flow in Plain and Reinforced Concrete Arches"*

J. R. Shank‡ (*Columbus, O.*—by letter)—Professor Probst has indicated a research field that has been entered by very few experimenters, His paper emphasizes the importance of the experimental side. In Europe this may be a proper emphasis, but in America where less stress is laid on mathematics in engineering education, the emphasis is, many times, better placed on the mathematical side. The two should go hand in hand, though in engineering fields the experimental must usually make the start. The field of plastic flow for sustained loading has been fairly well entered by the work of R. E. Davis[1] and others on the experimental side. Attention might well be now given to the mathematical.

A very important contribution to this has been given by W. H. Glanville[2] of England. For a concentrically loaded reinforced concrete column he gives this expression for the unit stress in concrete at any stage of plastic flow.

$$f_c = \frac{f_o}{e^{c/b}}$$

where f_o is the unit stress in the concrete immediately after loading or the elastic unit stress, e is the Naperian base, c is the plastic flow for plain concrete, and

$$b = \frac{A_c}{A_s}\frac{1}{E_s} + \frac{1}{E_c}$$

whose terms are those ordinarily in use in America. Charles S. Whitney has referred to this work and used this formula in a former paper.[3]

The writer of this discussion had some difficulty in satisfying himself of the accuracy of this formula, and he had occasion to read a

*Journal Amer. Concrete Inst., Nov.-Dec. 1933; *Proceedings*, Vol. 30, p. 137.

‡Research Professor, Eng. Exp. Sta., Ohio State University.

[1]"Flow of Concrete Under Action of Sustained Loads," R. E. Davis and H. E. Davis, Journal, Amer. Concrete Inst., Mar., 1931; *Proceedings*, Vol. 27, p. 837.

[2]"Studies in Reinforced Concrete." Building Research Technical Paper; No. 12, The Creep or Flow of Concrete Under Load, W. H. Glanville.

[3]"Plain and Reinforced Concrete Arches," Progress Report of Committee 312, Journal Amer. Concrete Inst., Mar. 1932; *Proceedings*, Vol. 28, p. 479, Charles S. Whitney.

discussion whose deductions indicated that it was incorrect. A fresh start was made from a different standpoint and the work in detail is here given.

From a study of much of the data on the plastic flow of plain concrete now available, it appears that an ordinary power formula similar to that of the parabola fits most cases.

$$y^a = Cx \quad \text{or} \quad y = Kx^{\frac{1}{a}}$$

The plastic deformation for unit stress y after a time x may be found for any concrete when the exponent $\frac{1}{a}$ and the constant K are determined. These may be found from plastic flow data by plotting on logarithmic paper deformations for unit unit stress as ordinates and time as abscissae. Where the straight line curve cuts the unit time line the value K may be read as an ordinate and the value $\frac{1}{a}$ is the slope of the line. The writer has found y in millionths of an inch per inch and x in days to work very well for ordinary data.

The plastic flow for reinforced concrete columns will be less than for plain columns of the same gross dimensions because of the reduction of the unit stress in the concrete due to the accumulation of stress in the non-plastic steel. This reduction of the unit stress in the concrete beginning immediately after the loading causes a further reduction in the plastic flow because of the elastic recovery due to this reduction.

Experiments have shown that within working ranges for concrete the plastic flow is directly proportional to the unit stress. Fundamental expressions for plastic flow can then well be expressed in terms of plastic flow for unit stresses of unity.

At any time during the plastic flow life of a reinforced concrete column under a load P we will have

$$P = A_c f_c + A_s f_s$$

After an increment of time x we have

$$P = A_c(f_c - f_{cc}) + A_s(f_s + f_{sc})$$

where f_{cc} and f_{sc} are the changes in unit stress for concrete and steel repectively for the time increment. This can be written thus:

$$A_c f_{cc} = A_s f_{sc} \quad \ldots\ldots (1)$$

We will now let s be the plastic flow in a *reinforced* concrete column from the initial loading to a time x. This value s also represents the change in the deformation in the steel which may be expressed as $\frac{f_{sc}}{E_s}$.

From (1) $\quad s = \dfrac{A_c f_{cc}}{A_s E_s}$

Now s is made up of the plastic flow from a varying unit stress f_c-f_{cc} for the time x and the elastic recovery $\dfrac{f_{cc}}{E_c}$, a negative value. We will now let y be the deformation that would have taken place but for the elastic recovery, or in other words, the deformation due to plastic flow as for a plain concrete column with a varying unit stress.

$$y = s + \frac{f_{cc}}{E_c}$$

$$y = \frac{A_c f_{cc}}{A_s E_s} + \frac{f_{cc}}{E_c}$$

$$f_{cc} = y \frac{A_s E_s E_c}{A_c E_c + A_s E_s} \qquad (2)$$

Now plastic flow in general is expressed:

$$y = K x^{\frac{1}{a}}$$

and

$$\frac{dy}{dx} = \frac{K}{a x^{\frac{a-1}{a}}}$$

is the rate of change at any time of the plastic flow for a unit unit stress. Multiplying this by the unit stress at any time gives the rate for that unit stress:

$$\frac{dy}{dx} = (f_c - f_{cc}) \frac{K}{a x^{\frac{a-1}{a}}} \qquad (3)$$

It will be noticed that f_{cc} is a variable whereas f_c is the unit stress immediately after loading and is therefore a constant.

Differentiating (2) with respect to y gives

$$\frac{df_{cc}}{dy} = \frac{A_s E_s E_c}{A_c E_c + A_s E_s} \qquad (4)$$

which represents the rate of change of the unit stress difference with respect to the plastic flow.

Multiplying (3) and (4):

$$\frac{df_{cc}}{dx} = \frac{A_s E_s E_c (f_c - f_{cc}) K}{a(A_c E_c + A_s E_s) x^{\frac{a-1}{a}}}$$

For convenience let

$$M = \frac{A_s E_s E_c K}{a(A_c E_c + A_s E_s)}$$

Then,

$$\frac{df_{cc}}{Mf_c - Mf_{cc}} = x^{\frac{a-1}{a}} dx$$

Integrating:

$$-\frac{1}{M}\log_e [f_c - f_{cc}] + C = ax^{\frac{1}{a}}$$

When $x = 0$, f_{cc} also equals zero.
Then,

$$C = \frac{1}{M}\log_e f_c$$

Then,

$$\log_e \left[\frac{f_c}{f_c - f_{cc}}\right] = Max^{\frac{1}{a}}$$

Putting in the original value for M,

$$\log_e \left[\frac{f_c}{f_c - f_{cc}}\right] = \frac{A_s E_s E_c K x^{\frac{1}{a}}}{A_c E_c + A_s E_s}$$

Using the values $p = \dfrac{A_s}{A_s + A_c}$ and $n = \dfrac{E_s}{E_c}$ we have

$$\log_e \left[\frac{f_c}{f_c - f_{cc}}\right] = \frac{p E_s K x^{\frac{1}{a}}}{p(n-1) + 1}$$

Now $\dfrac{f_c}{f_c - f_{cc}}$ might be written $\dfrac{f_c}{f_{cx}}$ where f_{cx} represents the unit stress at the time x.

Then,

$$\frac{f_c}{f_{cx}} = e^{\left[\frac{p E_s K x^{\frac{1}{a}}}{p(n-1)+1}\right]}$$

or

$$f_{cx} = \frac{f_c}{e^{\left[\frac{p E_s K x^{\frac{1}{a}}}{p(n-1)+1}\right]}}$$

From this the unit stress in any reinforced concrete column may be found from the elastic unit stress as commonly figured and the plastic flow property terms "a," the exponent and "K," the constant.

Glanville's formula may be changed to:

$$\log_e \frac{f_o}{f_c} = \frac{C}{b} = \frac{C}{\frac{A_c}{A_s E_s} + \frac{1}{E_c}} = \frac{C A_s E_s E_c}{A_c E_c + A_s E_s}$$

Now Glanville's f_o and $f_c = f_c$ and f_{cx} or $(f_c - f_{cc})$ above, respectively

$$\log_e \frac{f_c}{f_c - f_{cc}} = \frac{C A_s E_s E_c}{A_c E_c + A_s E_s}$$

Since

$$y = Kx^{\frac{1}{a}},\ y \text{ or } c = Kx^{\frac{1}{a}}$$

Then

$$\log_e \frac{f_c}{f_c - f_{cc}} = \frac{A_s E_s E_c K x^{\frac{1}{a}}}{A_c E_c + A_s E_s}$$

which is an equation given in the work of the writer just after the place where the value for M is substituted. This, therefore, offers a verification of the Glanville formula.

For example, a column 10 in. core diameter with 6 1-in. round bars will be taken. The concrete properties are $f_c' = 3000$ p.s.i., $E_c = 3{,}000{,}000$ p.s.i., and $n = 10$. The plastic flow properties are taken from the data by R. E. Davis; Series 7, 1:3:6 mix, air storage, 50 per cent Rel. Hum., and loaded at 28 days. The values for K and a are found to be 0.1225 millionths in. per in. and 3.6 respectively. A full working load of 159,500 lb. will be considered from which f_c will be 1320 p.s.i. The stresses in the concrete and the steel as well as the flow contraction will be computed for 365 days.

$$f_{cx} \text{ (the unit stress in the concrete at } x \text{ days)} = \frac{f_c}{e\left[\frac{p E_s K x^{\frac{1}{a}}}{p(n-1)+1}\right]}$$

The exponent for $p = 0.06$, $K = 0.000{,}000{,}122{,}5$, $x = 365$, $E_s = 30{,}000{,}000$ p.s.i. and $n = 10$ is computed to be 0.738 and the denominator entire is: 2.092

$$f_{cx} = \frac{1320}{2.092} = 631\ \text{\#}/\square''$$

$$631\ \text{\#}/\square'' \times (73.8\square'' = A_c) + f_{sx} \times (4.71\square'' = A_s) = 159500\ \text{\#} = P$$

$$f_{sx} = 24000\ \text{\#}/\square''$$

$$\frac{24000 - 13200}{30{,}000{,}000} = 0.00036''/'' \text{ contraction}$$

or 0.043 inches in ten feet.

This discussion is concluded except for the author's opportunity for closure in the November-December JOURNAL.

Discussion of a paper by J. W. Beretta:

"Rigid Frame Concrete Bridges"*

K. Hajnal-Konyi, (Darmstadt, Germany—by letter, translated by Inge Lyse). With the conviction that it is of advantage to compare the experiences of different countries, I am taking the liberty of referring to an article on reinforced concrete frame bridges of large spans which I recently published.[1] Its purpose was to present a tabulation, grouped by countries, of all reinforced concrete girder bridges in the world with spans of more than 30 m. (98.4 ft.). This article developed that there was no clear definition of the apparently so simple element of statics as the "beam," which would stand out against other types of construction. I have made use of such a definition which satisfied all types of girder bridges, but will omit further discussion of it here. It may be of interest to note the fact that my list contained 62 bridges of which Germany (21) and Hungary (20) each contributed about one-third. The remaining 21 bridges were spread over 11 countries, among which the United States had 3. I have later learned of 12 more bridges with spans exceeding 98.4 ft. (2 in U. S.), the list of which will soon be published in *Beton und Eisen.*

The table contains all information of significance, including year of construction and both longitudinal and cross-sectional sections to the same scale so that the size relations may readily be seen. The years of construction show that Hungary was the first country in which reinforced concrete girder bridges of large spans were used. Hungary had a bridge of 126 ft. span in 1908 and at the outbreak of the World War it had 16 large span bridges, of which 15 were in those sections which were lost to Hungary through the peace conference. The first girder bridge in Germany with a span of 105 ft. was constructed in 1911. This is similar to the Martinez bridge described by Beretta with the exception of the use of two hinges. Germany at present has 26 reinforced concrete girder bridges with spans exceeding 98.4 ft.

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*Presented at the 29th Annual Convention, Chicago, Feb. 21-23, 1933. Journal, Amer. Concrete Inst., Jan.-Feb. 1934; *Proceedings* Vol. 30, p. 196.

[1]*Beton und Eisen*, No. 23-24, 1933.

Since

$$y = Kx^{a}, \; y \text{ or } c = Kx^{\frac{1}{a}}$$

Then

$$\log_e \frac{f_c}{f_c - f_{cc}} = \frac{A_s E_s E_c K x^{\frac{1}{a}}}{A_c E_c + A_s E_s}$$

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The table also shows the different types of statically determinate and indeterminate systems used. The statically determinate system is not used in European countries for the sake of simplifying the analysis, but for its ability to take care of uneven settlement, temperature changes and shrinkage. The illustrations give a clear picture of the statical system as well as the cross-sectional arrangements. The modern trend is towards the use of flat slab construction which has the advantage of reducing the number of main girders and also reducing the total dead weight of the bridge.

The reinforced concrete bridge of largest span, (223 ft.) is in Brazil. But the limit for this type has not as yet been reached as indicated by the competition project for the Dreirosen[2] bridge in Basel, Switzerland, where it was shown that reinforced concrete girder bridges could compete with steel girder bridges for spans up to 350 ft.

The railroad bridge of largest girder span was built in Algiers by a French engineer. The span is 121.5 ft. The further development of this type of construction is principally dependent upon the production of cement of less shrinkage and greater strength.

F. G. Brouyakine† (Moscow, U. S. S. R.—by letter). Mr. Beretta deals with the advantages of reinforced concrete rigid frame bridges in comparison with the use of beams on separate piers. I should like to record something about our experience with bridges of this type in the U. S. S. R. I commend the author when he says that "it is not the intention of the writer to present structural continuity as a design method of limitless possibilities in reinforced concrete as a panacea for the weaknesses of other structural materials." Rigid frame bridges have their advantages and disadvantages—the main advantage being economy. Under the conditions of the U. S. S. R. the rigid frame type sometimes will be as much as 30 per cent cheaper than a bridge with separate beams. They, however, have all the disadvantages of statically indeterminate systems, so I think the author is wrong when he says that rigid frame bridges were not built because of the laziness and conservatism of engineers. The rigid frame type was not built because engineers gave no credit to the benefit to be derived from the use of reinforced concrete in the construction of this type of bridge.

The main defect of rigid frame bridges is their sensibility to the unequal settlement of the piers. The settlement of the pier of 1.0-1.5

[2] *Beton und Eisen*, No. 13-14, 1931.
† Bridge Engineer of Central Office of new railway construction, Peoples Commissariat of Transport

c. m. (3/8—5/8 in.) causes great overstresses and may destroy the serviceability of the bridge. Therefore rigid frame bridges must be built on good soil having a compressability equal for all its piers. In the case of bad ground most of the economy will be lost in the additional expense of the foundation. Simple reinforced concrete beam bridges are not damaged in the same conditions and can be built on somewhat less secure foundations.

The natural limit for the building of a rigid frame bridge is the height of the embankment. The bridge of this type is economical when that height is no more than 16-18 meters. The higher embankment requires longer spans, but the span of the railroad frame bridge for modern loading is economically limited to about 20 meters.

In places where the bridge spans a flow of heavy ice, multispan frame bridges are not allowable. Rigid frame bridges have additional temperature and shrinkage strains and stresses. These stresses can be very high when the design of rigid frame bridge is not satisfactory. That is why the projecting and building of such bridges requires much more care. Rigid frame bridges are not a panacea for the small bridge problem. In spite of the disadvantages of this type of bridge their possible economy makes it necessary to consider them. Their development might well have worked out typical projects. Correctly applied standards could eliminate the errors committed by inexperienced designers.

I think the author's opinion of possible economy to be wrong. By projecting continuous and discontinuous typical railway bridges we have concluded that continuity does not give noticeable economy. The continuous beams are unprofitable in many cases. If also we consider that continuous beams are very sensible to the settlement of the piers the possibility of their use must be less. In the practice of railway building in the U. S. S. R. continuous beams have been little used for at least 15 years.

Rigid frame bridges in the U. S. S. R. are now in most cases on typical projects, which are worked out for height of the embankment of from 6 to 16 meters, and for spans from 8 to 16 meters with many combinations of these figures. Calculations for rigid frame bridges in the U. S. S. R. include besides dead and live load, centrifugal forces, temperature changes, and shrinkage.

Below is some information about the calculation required:

The coefficient of impact has a value for railways loading—

a) for span length L equal to or less than 5m. (16.5 ft.), the impact coefficient, C = 1.3
b) for L between 5 and 20 m. (16.5-66.5 ft.), C = 1.2
c) for L greater than 20 m., C = 1.1

The shrinkage is calculated as a lowering of the temperature on 10°.

The allowed stresses on concrete are based on the results of tests of 20-c.m. cubes, 28 days old.

The allowed stresses are the following for railway bridges:

1. Direct compression 0.35 R 28
2. Compression at bending 0.40 R 28
3. Compression at bending with calculation of force of wind or temperature with shrinkage or the brake force—0×45 R 28
4. The same but with consideration of the wind and temperature with shrinkage—0.55 R 28
5. The same with consideration of all forces 0.60 R 28
6. The maximum cleaving stresses in the case of full reinforcement 0.092 R 28
7. The compression and tension of the steel—1300 K. per cm² (18,500 p.s.i.)
8. The same but with consideration of wind or temperature—1450 K. per cm² (20,000 p.s.i.)
9. The same but with consideration of temperature and wind—1550 K. per cm² (22,000 p.s.i.)
10. The same in case of calculation of all forces—1650 K. per cm² (23,500 p.s.i.)

SOME STRUCTURAL REQUIREMENTS

The beams in all cases must have stirrups on centers no more than ¾ of height of the beam and no more than 50 cm. (20 in.) The number of steel rods which embrace one stirrup cannot be greater than:

for zone of tension—5 rods $d \leq 22$ mm. 4 rods for $d > 22$ mm.
and 3 rods for zone of compression.

The long cantilevers which end in the embankment are not recommended. The limit span of the cantilever is considered ⅕ of the next span of the frame.

The calculation of changes of moment of inertia is obligatory. The difference in relation of stiffness of beam and column applied in calculation and that after finishing the design cannot exceed 20 per cent.

In reference to projects mentioned by the author.

Martinez street bridge: This, which the author considers a perfect example of a structure has in our opinion some structural defects. For instance: the ribs and columns have no stirrups. Temperature effects on stresses would be very great. The connection of beam and column is unsatisfactory because this connection is made by very small radius.

Garden street over Comal river: The main defect of this bridge is that the beam of the bridge has a free leaning on the abutment. In the event of unequal settlement of the pier and abutment the cantilever will have considerable additional stresses.

The reinforcement is not so good. The columns have no stirrups. The beams have no retorted (skew, slanting) rods. The rods of tensile reinforcement end in zone of tension; it is in our opinion not to be recommended, especially when the lower surface of the rib is not horizontal as in this case.

I cannot agree with author that progress of constructing of small bridges can be reached only by using statically indeterminate systems. Greater results could be attained by a correct and economical design of simple statically determinate typical beams.

This discussion is concluded except for the author's opportunity for closure in the November-December JOURNAL.

Discussion of Paper by J. Morrow Oxley:

"The Toronto Building By-Laws"*

Eugene Mirabelli† (*Boston, Mass.*—by letter)—The writer is particularly interested in the part of the proposed revisions of the By-Law which deals with reinforced concrete slabs supported on four sides. It is pleasing to note the efforts of the committee to take into consideration the strengthening effect of "plate action" in this type of construction. Present American practices as expressed in the requirements of municipal building codes, standard specifications for concrete construction, and in textbooks on reinforced concrete, neglect entirely the torsional resistance of mutually perpendicular strips of slab and the consequent reduction in slab bending moment. The German Regulations of 1925 prescribe the method of H. Marcus, by which bending moments are corrected for "plate action" by modifying factors which vary with the number and location of restrained sides in the panel. The method of H. Marcus is also being recommended, for approximately square panels, by the advisory committee reporting on amendments to the London Building Act. Although American building codes have not yet followed this liberal tendency there are indications that committees which are now drafting revisions are considering the possibility of being less conservative with two-way slabs.

The proposed changes will bring into greater evidence the economy inherent in this type of construction and may lead to an increase in its use. In general, "plate action" leads to a decrease in the required thickness of slab and to an increase in the size of supporting beams. The latter is due to the fact that the slab, if freely supported, tends to rise away from the supports at the corners and produces a concentration of load on the beams near mid-span. Since the beams are deeper than the slab, they can resist bending more efficiently. The greatest saving in material occurs when the slab is supported on masonry walls and no beams are required.

The proposed equation for the maximum positive moment in the slab of a freely supported square panel, $M = \frac{wl^2}{24}$ is based on the

*Journal, Amer. Concrete Inst., Mar.-Apr. 1934, *Proceedings*, Vol. 30, p. 345.
†Assistant Professor of Structural Design, Massachusetts Institute of Technology.

assumption that the corners of the slab are anchored against uplift. With free corners, both loading tests and analysis show that the equation would be more nearly, $M = \frac{wl^2}{21}$. Corner anchorage forces, therefore, are required, each equal to about one-sixteenth of the total panel load.

The proposed equation for bending moment in the short direction of a freely supported rectangular slab, $M = \frac{wl^2\,(2k - 1)}{24}$, seems to be unnecessarily conservative for values of the ratio, k, which exceed 1.5. When the ratio, k, is 2.0, this equation gives a moment of the same magnitude as that required for a one-way slab in the short direction, and at the same time provision must be made for a moment, $M = \frac{wl^2}{24}$, in the long direction. The proposed equation yields moments for the short direction which exceed corresponding moments obtained by precise methods by Timoshenko, Pigeaud, and Marcus, by about 30 per cent when the ratio, k, is 2.0, and by about 15 per cent when the ratio is less than 1.5. It seems that the percentage margin over precise values should decrease as the ratio, k, increases, since with higher values of this ratio, the relative effect of "plate action" is diminished and the attending uncertainties are also diminished.

The bending moments in the supporting beams resulting from the loading distribution suggested in Fig. 2 (p. 357), are very nearly the same as those obtained by more precise methods. The actual beam loading varies more nearly as the ordinates to an ellipse. The beam end shears obtained by use of Fig. 2 will be about 15 per cent too small for a square panel.

It is difficult to cover all cases of continuity in slabs by applying a simple percentage reduction to bending moments determined for a simply supported panel. The ratios presented in the table on page 357 can give only a rough approximation to the actual bending moments. For example, consider the case of a square panel which is fully continuous on two opposite sides and freely supported on the other sides. Due to the increased stiffness in the continuous direction, the load transmitted in that direction is increased to about $\frac{5}{8}$ of the total panel load. The maximum positive bending moment in this direction is about 85 per cent of the moment for a freely supported panel instead of the 66⅔ per cent given in the table. For the freely supported direction there is a reduction to 75 per cent. It is not clear

from the table if it is intended that the reduction to 66⅔ per cent be taken for both directions or only for the continuous direction. However, there is an actual reduction in bending moment in the freely supported direction as well as in the continuous direction.

There is no mention of how slab shears are to be calculated, although it may be implied that shears may be computed from the loading distribution shown in Fig. 2 (p. 357). M. Pigeaud (*Annales des Ponts et Chaussees*, March, 1929) recommends the approximate equations,

$$V_1 = \frac{k}{2k + 1} \cdot wl \quad \ldots\ldots (1)$$

$$V_s = \tfrac{1}{3} wl \quad \ldots\ldots (2)$$

in which V_1 = shear at middle of long side (lb. per lin. ft.)
V_s = shear at middle of short side (lb. per lin. ft.)
(Other symbols are same as used by the author.)

For freely supported slabs, the writer suggests the equations,

$$V_1 = (0.25 + 0.11\,k)\,wl \quad \ldots\ldots (3)$$

$$V_s = (0.44 - 0.08\,k)\,wl \quad \ldots\ldots (4)$$

CONVENTION DISCUSSION

F. E. Richart: (In response to question) I might comment on the reason for Westergaard's reduction of moment in slabs and not in the beams supporting them. We know that in tests slabs usually show high strength and have explained that phenomenon, especially with regard to flat slabs and two-way slabs, by the fact that the slab is lightly reinforced so that the concrete is effective to some degree in tension and by the fact that the slab is reinforced by a network of steel so that if it is overstressed at any local point, other parts of the slab will carry the load with no great damage to the structure. These are advantages of the slabs which do not apply to the supporting beams. Furthermore, if a slab is supported on beams but not attached to them and is subjected to the same loads and moments and shears as if it were attached, there is a tendency for the corners of the slab to curl up. In an ordinary structure these corners are held down to the beam; therefore the slab exerts upward forces on the beams near the ends and consequently there is more than a hundred per cent of downward load at intermediate points. The result is that the beams supporting the two-way slab of course do not have a uniform loading; there is a concentration of load toward the middle of the beam and therefore more than the normal moment produced in them. Therefore if we use the bending moment without reduction, we probably have already allowed for some increase in the load produced by this non-uniformity

of load. I think those are some of the considerations that guided Doctor Westergaard in reducing slab moment, but in not giving the beams the benefit of such reduction. I think also that in his assumption of load on adjacent panels, the effect is more on the beam than on the slab. That is another reason for not reducing the beam moment.

Mr. Oxley: In Westergaard's study, he comments on that point as an explanation of the fact that the beam moments look relatively high, but in arriving at his beam moment, he takes the total of WL over 8, deducts from it the reduced moment found in the slab and the balance he calls the beam moment, but the thought was that that reduction is going to apply to some extent, an unknown extent it is true, to the beam, that the load from the slab will tend to be carried perhaps not directly to the corners but in lines approximating diagonals to some extent; in other words, there will be some radial distribution of load from the slab that will not necessarily be all a rectangular distribution to the beams in the two directions. If the moment in the slab may be reduced by the 28 per cent, it would appear that the reason for that reduction, or part of the reason at any rate, is that your slab moments are not acting on rectangular lines altogether, that they are distributing radially as well; any radial distribution they get will tend to lighten the moment in the beam; in other words, a load from somewhere near the center of the slab will be carried to some point on the beam nearer the support than the center of the beam. Now I am free to admit that I cannot back that reasoning up by mathematical analysis, but on the basis of Westergaard's own reasoning, which incidentally he does not attempt to back by exact mathematical analysis on that point, it seems to me it is reasonable to allow some distribution. In other words, the total reduction we get in the case of a flat slab is 28 per cent; in the case of a two-way slab we have a situation intermediate between a flat slab and a one-way construction. Surely then the total moment which has to be considered should be somewhere intermediate between these two. To find the exact position in between those two, is very much beyond me, at any rate, but it seems sound by analysis that it should be somewhere between the two. The total moment in the long beam which we are proposing is about 11 per cent less than Westergaard proposes. It is a relatively small difference; that eleven per cent does not seem an unreasonable amount to take, considering the 28 per cent reduction we allow in the case of the flat slab. As I say, I cannot attempt to support it by any exact mathematical analysis; it is only a case of analogy. In regard to the curling up of the corners, I did not take time to read through the causes

covering that, but we have provided that the corners shall be securely anchored down and that the reduction in slab reinforcement shall occur only parallel to the continuous edges of the slab, adjacent and parallel to continuous edges of the slab; parallel to the non-continuous edges, no reduction is provided.

This discussion is concluded except for the author's opportunity for closure in the November-December JOURNAL.

Discussion of the Paper by H. M. Westergaard:

"Stresses at a Crack, Size of the Crack, and the Bending of Reinforced Concrete"*

Inge Lyse† and *D. M. Stewart*† (*Bethlehem, Pa.*—by letter): Professor Westergaard has in this excellent treatise on stresses at a crack raised a question of fundamental interest to the engineering profession. This question is on the stress condition at the root of a crack in the case of a reinforced concrete beam with a crack extending to the neutral axis. To study experimentally this particular problem, the specimen shown in Fig. 1 was cut from a sheet of bakelite, 0.264 in. thick, and subjected to photoelastic analysis by means of the Winkler-Zeiss apparatus, recently added to the equipment of the Fritz Engineering Laboratory. A slot which was intended to simulate a crack was extended up to the computed neutral axis of the specimen, leaving a section of the material to act as the reinforcement to resist the tensile force. This beam was subjected to direct bending, and the photoelastic pattern shown in Fig. 2 was obtained. This figure shows that no stress exists at the root of the crack. Furthermore the first fringe shows a dip next to the cracked section, indicating some readjustment of the stress distribution above the neutral axis at the crack. This is better illustrated in Fig. 3 which shows the evaluation of the stresses. The crack has disturbed the straight line stress distribution at the vicinity of the crack, and a stress diagram similar to that presented in Fig. 3 of Professor Westergaard's paper is obtained.

This simple photoelastic study of a bakelite specimen illustrates how rather difficult theoretical problems may readily be subjected to direct observations. In the particular case analyzed, since the shear across the central section was zero, the photoelastic fringes gave a direct measure of the normal stresses at this section. Although the evaluation of the principal stresses P and Q for a more complicated type of test conditions is not as yet a very simple matter, the partial analytical and partial graphical method of solution serves well for

*Journal. Amer. Concrete Inst., Nov.-Dec. 1933; *Proceedings* Vol. 30, p. 93.

†Respectively, Research Associate Professor and Research Fellow, Fritz Engineering Laboratory, Lehigh University.

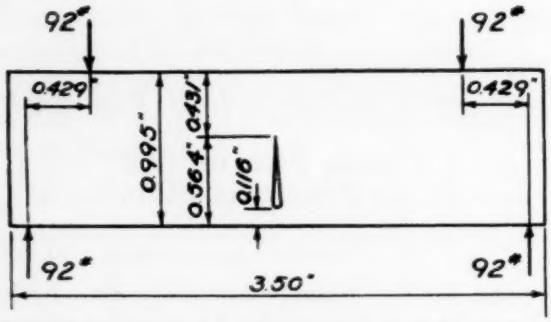

FIG. 1—BAKELITE MODEL OF A CRACKED BEAM WITH TENSILE REINFORCEMENT

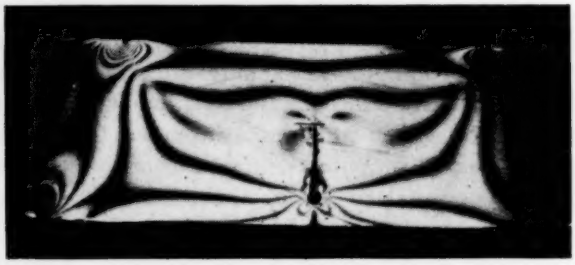

FIG. 2—PHOTOELASTIC FRINGE PATTERN OF MODEL UNDER UNIFORM BENDING MOMENT OF 39.45 IN.-LB.

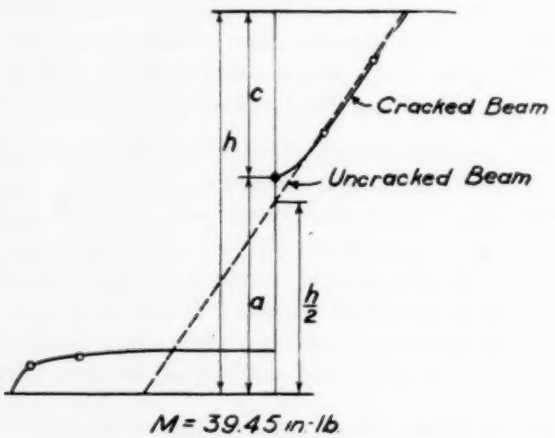

FIG. 3—STRESS DISTRIBUTION IN A CRACKED BEAM UNDER UNIFORM BENDING MOMENT

their evaluation and with some practical experience gives the solution with a reasonable amount of work. Further simplification is obtained when soap film experiments are used for observing the $P + Q$ values.

CONVENTION DISCUSSION

S. C. Hollister: May I discuss Professor Lyse's last slide? (Fig. 2 foregoing) I wish to point out two or three essential things in the photo-elastic analysis. Professor Lyse spoke of the spacing between successive fringes of the same color. This spacing is not a constant for successive fringes and it may be that where the spacing is different in amount from one fringe to the next, that is, the successive increment between fringes of equal color, it does not necessarily represent a state of uniformly varying stress. Furthermore, the picture shows stress differences and not the stress under that section. We may have conditions at a neutral axis where the stress in the longitudinal direction is zero but where the stress in the vertical direction is not zero, and hence we would not be able to see the neutral axis, and for that reason there are portions where the neutral axis, as we would think of it in ordinary terms, so far as the flexure alone is concerned, is not visible. As Professor Lyse has pointed out, the stress at this point does not indicate a stress concentration of the nature indicated here.

Prof. E. K. Timby: I should like to support Professor Lyse in his statement that his equal differences between the fringe line might represent equal differences in stress. I have had occasion to measure the fringe value and have found that in plotting a fringe ordinate against stress in pounds per square inch, that at least for the specimen I was using, a thin bakelite specimen of amber color, I got practically a straight line. I should also like to ask Professor Hollister, anticipating he does intend to continue his study and evaluate P and Q separately, just what method he intends to use in finding P plus Q from his values of P minus Q?

Professor Hollister: First, with regard to the constant stress differences between successive fringes, I should like to refer to a series of beautiful studies made by R. V. Baud, recorded between the years 1929 and 1932 in the *Journal of the Optical Society of America*, in which this matter of constants in the increment of stress between successive fringes is discussed at length. The second point, how shall we measure the separate values of P and Q: It is hoped that some information may be had from the study of some rubber specimens, the properties of which have been fairly carefully studied to determine the change in thickness of the assembly in the direction in which the light is passing

in the photo elastic specimen, which change in thickness would be proportional to the stress sum. Having that value, knowing the stress sum and the stress difference, then this problem perhaps could be worked out. The other method which has been tried but so far without success, is by means of a method proposed by Mesnager, in France.

Professor Westergaard: May I call attention to the experimental method of finding the sum of the principal stresses at each point by use of a soap film?* The soap film stretched over an opening with distorted edge is a splendid machine for solving certain equations, but it is easier to say "use a soap film" than actually to do it. During visits at the University of Michigan I have had the opportunity to observe the progress made first by Dr. P. A. Cushman and later by Dr. E. E. Weibel† in the technique of the soap film. By patience and perseverance, working during the no-man's hours of the early morning, Dr. Cushman arrived at a technique which Dr. Weibel to some extent inherited. So when Dr. Weibel reported the results of studies of photo-elasticity in connection with the soap film, photo-elasticity furnishing the difference of the principal stresses and the soap film their sum, he was able to point with satisfaction to the ease and speed with which the soap film can be used. This technique may now be considered to be available, and I believe that the method will be found preferable to the graphical integrations and the method based on measuring the changes of thickness.

May I be permitted to make one more comment? It is evident that the theory which has been presented is in a stage of incompleteness and it is a question whether one should present something as incomplete as this. I wish to comment on what the Concrete Institute has done. I submitted the paper in September and the Institute published it in the November-December JOURNAL, and that is an unusual speed in this country. This means that the Concrete Institute is functioning as a forum for technical discussions. It does not wait for two years before a paper is published, and the investigation does not have to be finished into such shape that it can go down as something classical. This report of work in progress has been made possible by the Concrete Institute, and I express indebtedness to the Institute for having given space to the paper.

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*Originated by J. P. Den Hartog, Zeitschrift für angewandte Mathematik und Mechanik, Vol. 11, 1931, p. 156.

†E. E. Weibel, Studies in Photoelastic Stress Determination, Transactions of the American Society of Mechanical Engineers, August, 1934.

Commander Ben Moreell: I will probably be "sticking my neck out" by getting into this very learned discussion. The name of Mesnager was mentioned. I had the honor of listening to a course of lectures by him, in which he explained in great detail his pioneering work in photo-elasticity. He has a conception of the nature of the failure of concrete based on his photo-elastic work, which perhaps would be interesting to those of us who are working with concrete. I will not attempt to go into detail in this explanation; it is probably an old story to some of these gentlemen who have been working with photo-elasticity. His conception of the failure of concrete in compression is that it is really a tension failure and that the failure results from the existence of very small cavities in the body of the concrete giving a concentration of tensile stress at the edges of those cavities, some of which might be almost microscopic. This has a practical application because it might be related to the well known fact that as we increase densities and reduce the sizes of those microscopic cavities, we increase the compressive strength of the material.

This discussion is concluded except for the author's opportunity for closure in the November-December JOURNAL.

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's corps of Reviewers.

Studies of Italian pozzuolana and pozzuolanic portland cement

S. NAGAI and I. YOSHIDA. *Cement and Cement Manufacture*, Vol. 7, No. 8, Aug. 1934, p. 254-57. Reviewed by J. C. PEARSON.

This short paper is worthy of note only as a source of chemical data on Italian and Japanese pozzuolana and pozzuolanic cements. Compressive strengths of plastic mortar cylinders, 2 cm. diam. by 3 cm. long, obtained from the various combinations tested, are given without comment

Deformations in the interior of concrete structures

MARIANI, *Journal L'Entreprise Francaise*, June, 1934, p. 13. Reviewed by P. H. BATES.

The author, although of the faculty of the Polytechnic Institute of Milan, presents a very interesting discussion of the theories, mostly French, which have been advanced regarding expansion, contraction, flow, fatigue, creep, etc., due to the hardening, loading, and temperature variation of concrete. The article does not lend itself to abstracting, but it should be of value to those interested in this subject.

Recent innovations in the manufacture of asbestos-cement

E. LECHNER; *Cement and Cement Manufacture*,, Vol. 7, No. 6, June, 1934, p. 178-82. Reviewed by J. C. PEARSON

This paper refers to the wet or Hatschek process of making asbestos-cement shingles, wall-boards, etc. It should interest the manufacturers of these products, and those who desire to be generally familiar with the more important special uses of portland cement. Sources and properties of asbestos-fibre are discussed, as well as recent improvements in manufacturing equipment. The process of coloring with ferrous and other salts is briefly described, also the manufacture of flat and corrugated sheets, glazed sheets and asbestos-cement pipes.

Theoretical study of the vibration of concrete

COYNE, *Journal L'Entreprise Francaise*, June, 1934, p. 10. Reviewed by P. H. BATES.

The author gives in rather too meager detail a method for detecting the vibrations, and their amounts, imparted to concrete by vibrating machines. It is based upon the piezoelectric properties of quartz. If a quartz crystal is properly cut and two opposite faces are submitted to alternate pressure or tension, a current is generated. This current is amplified by an amplifying tube and then passed into a "sort of galvanometer with an oscillating mirror." A beam of light reflected from the mirror draws a curve on a photographic paper. The quartz is placed in a small metal

capsule, one face of which imparts the vibration to the quartz. The capsule can be placed anywhere within the concrete and the vibrations noted. The necessary lead wires bring the generated current from the quartz to the amplifying and recording device. The apparatus is so designed that it can be moved about from place to place in the concrete during the vibration.

Thermal studies of cement

EDMOND MARCOTTE, *Le Ciment*, May, 1934, No. 5. Reviewed by P. H. BATES.

Presents results obtained at the laboratory of Ponts et Chausses in observing the heat developed during the setting of cement and certain other of its physical properties. Some correlation of composition and heat is also presented. The apparatus used was a simple, well-insulated box holding the specimens during the early (24 hour) stage of setting. Temperatures were recorded by an inexpensive recording thermometer. The author indicated that from a curve obtained by the plotting of the rise of temperature against time, it is possible to obtain the initial set with "precision," and somewhat less precisely the final set. Plotting the temperature rise against the compressive strength of a 1:3 plastic mortar, it was found that from the area between the curve and the abcissa at the maximum temperature and the ordinant of initial temperature and the strength of a sufficient number of cements, it was possible to derive a factor by which the strength could be determined from the temperature.

Lightweight concrete construction

J. SINGLETON-GREEN, *The Structural Engineer*, Vol. 12 (New Series), No. 4, Apr. 1934, p. 186. Reviewed by N. M. NEWMARK.

This paper gives a description and brief discussion of some of the ways in which lightweight concrete construction may be secured. The weight of concrete can be reduced by omitting some of the material, as in ribbed slabs or walls, or by reducing the unit weight of the concrete either by using light weight aggregate or by introducing voids by means of mechanical or chemical processes. In some cases a reduction in weight may be obtained by using a stronger concrete with higher working stresses.

Lightweight aggregates include natural aggregates such as pumice, and artificial aggregates such as coal residues, blast-furnace slags, sawdust, and burnt clay or shale. Concrete with a great number of voids, or cellular concrete, may be made by introducing bubbles of air mechanically or by adding chemicals which react with each other or with the cement to evolve gas.

The chief difficulty in producing light weight concretes has been not to get a light weight alone, but to get a light weight together with a reasonably high compressive strength.

Special cements

C. R. PLATZMANN, *Cement and Cement Manufacture*, Vol. 7, No. 3, Mar. 1934, p. 61-67. Reviewed by J. C. PEARSON.

Describes briefly the production and characteristics of existing cements other than portland. The list includes: Rapid-hardening or high-early-strength cement, white portland, Roman cement, Selenitic cement (hydraulic lime with gypsum admixture), natural cement, grappier cement (France), ore cement, iron portland and blast furnace cements, pozzuolanic cements, soliditit cements, and aluminous, or high-alumina cements. With respect to those less familiar to American readers, it may be stated that Roman, natural and grappier cements are similar in composition, and somewhat akin to portland; ore cements are true portlands but with the usual percentages of

iron oxide and alumina reversed—somewhat like the Boulder dam cement, but generally higher in iron and lower in alumina; iron portland and blast furnace cements are portland ground with different percentages of blast furnace slag; pozzuolanic cements are lime ground with natural or by-product materials high in acid-soluble silica; soliditit cements are portland ground with certain types of calcined siliceous materials, like granite, basalt, etc. Chemical analyses and test data are given for the more important of these special cements.

Concrete in sacks for breakwater and lighthouse foundations —anchorages of pre-stressed steel bars

M. Petry (Engineer of Ponts et Chaussees); *Les Annales des Ponts et Chaussees*, 1934—II. Mar.-Apr., p. 234. Reviewed by B. Moreell.

In this article the author lists the advantages of using concrete in large sacks to provide foundations for massive structures on rock where the structures are exposed to wave action. The advantages are (a) sacks of concrete once placed are immovable, (b) they offer more resistance to wave action than a rock fill, (c) they do not require prior leveling of the bottom as do monolithic blocks and (d) they are much cheaper than masonry constructed under compressed air.

He proceeds to describe the advantages of anchoring works of this kind to the underlying rock stratum by means of steel tie bars or cables which are placed in an initial state of tension. This system has actually been applied to the construction of a dam in France.

The theory of the inter-action of concrete and pre-stressed steel bars was first enunciated by M. Freyssinet and a complete exposé of this theory was published in *Science et Industrie*, January 1933. A translation of this theory has been made by the reviewer and a limited number of copies are available for distribution.

Heat of hydration of cements for Boulder dam

Thomas J. Noland, Jr., *Civil Engineering*, Vol. 4, No. 7, July, 1934, p. 365. Reviewed by J. R. Shank.

Reports part of the work done by the U. S. Bureau of Reclamation at the University of California for the Boulder dam project. The author describes the three calorimeter procedures, the adiabatic method, the high-insulation method, and the heat-of-solution method, which were used. Each is given in considerable detail, together with sketches and photographs. In general, it was found that 60 per cent of the heat is given off the first day, 90 per cent in the first 28 days, and practically 100 per cent in 180 days. Other observations are quoted:

"1. The chemical composition of cement has a marked effect on the amount of the heat of hydration. It is possible to reduce the heat of hydration below that of normal portland cement by proper regulation of the chemical composition.

"2. In general, at 28 days, under conditions of mass curing, each 1 per cent of tri-calcium aluminate produces approximately 2.5 g-cal; each 1 per cent of tri-calcium-sulfate produces approximately 1.1 g-cal; and each 1 per cent of di-calcium sulfate and tetra-calcium-alumino-ferrite produces approximately 0.45 g-cal.

"3. Increasing the fineness of the cement increases the heat of hydration at the early ages.

"4. Increasing the water-cement ratio increases the heat of hydration at the early ages.

"5. Cements that show a great deal of strength in proportion to the amount of heat generated are those which are low in tri-calcium-aluminate.

"6. Heat of hydration is increased by lowering the initial temperature of curing."

Some new relations bearing on concrete mixtures

Public Roads, May 1934, Vol. 15, No. 3. Reviewed by F. H. JACKSON.

Reports tests to study effects of variation in the proportions of the solid ingredients of concrete and its water content upon the quality of the resulting mixture as measured by its density and strength.

The absolute volume basis of proportioning was used; basic water content fixed by density determinations and the resulting concrete-voids curves constructed from these determinations. All concrete specimens, for both density and strength determinations were molded by mechanical means.

The tests developed indications of certain relations existing in concrete mixtures:

1. For a particular combination of fine aggregate and cement a relation exists between the amount of coarse aggregate in the mixture and (1) the amounts of water required as the basic and various relative water contents and (2) the corresponding total voids in the mixture.

2. Likewise, for a particular coarse aggregate content, a relation exists between the ratio of the amounts of fine aggregate and cement in the mixture and (1) the amounts of water required as the basic and various relative water contents, and (2) the corresponding total voids in the mixture.

3. For each relative water content, using the same kinds of materials, the slump of every concrete mixture will be the same regardless of the proportions of cement and aggregate used in it.

The article describes the methods used and contains the basic data obtained from the tests.

Pumping concrete for dam construction

E. W. SEEMAN, *Civil Engineering*, Vol. 4, No. 8, Aug. 1934. Reviewed by J. R. SHANK.

A special pump and 7-in. pipe line were used on the construction of Dam No. 5 on the Mississippi river about 100 miles below St. Paul to convey a harsh 1½-in. slump concrete from the mixer to placement. The cement was required to have not less than 15 per cent retained on a 200-mesh sieve. The aggregate was as coarse as 2 in. The yield in the field was 4.5 to 4.6 bags of cement per cubic yard. The strength attained was as high as 4000 p.s.i. Vibrators were used for placing.

Reciprocating pumps in pairs were used at the mixers to force the concrete through the pipe lines. Each cylinder had a bore of 7½ in. and a stroke of 12 in. They operated at about 50 strokes per minute. The valve ports were almost as large as the cylinder bore. The valves were made with springs so that they did not have to close completely each time. A piece of aggregate between the valve and its seat did not put the pump out of action. The constriction at the valve port served to have the concrete make its own plug. One pump could force this concrete through 1000 ft. of straight pipe. Pipe bends were made with long radii.

It was found safe to interrupt pumping for as much as half an hour but it was found generally good to give a few strokes every few minutes during the shut down. The pipe sections were 10 ft. long and the couplings were so made as to be easily and quickly connected and disconnected.

At the end of a run a section or two of pipe near the pump was removed and a dumb-bell shaped, rubber packed "go-devil" was inserted at the pump end. The pipe was reconnected and water was run through the pump. This go-devil was forced through the pipe, thus clearing it of concrete and washing both pump and pipe line.

Thin shells in the shape of hyperbolic paraboloids

BERNARD LAFFAILLE, *Le Genie Civil*, Vol. 104, No. 18, p. 409-410. Reviewed by R. L. BERTIN.

Comments on an article in *Le Genie Civil* of February 25, 1933 by Mr. Aimond, Chief of the Research Bureau of the Air Ministry of France, which reveals the results of studies on the subject of thin shells curved in opposite directions for use in the construction of roofs for hangars, briefly as follows:

After considering all factors, such as lightness, large spans, resistance to wind and snow loads, and the already existing types, such as cylindrical vaults, spherical domes, ribbed vaults, conoids of the 3rd order, the conclusion was that the most interesting shape was the hyperbolic paraboloid.

The author calls attention to a patent granted January 21, 1931 on such a surface, defined therein as being generated by a straight line remaining parallel to a directional plane and in contact simutlanaeously with a straight line and a curve, neither of which are in planes parallel to the directional plane.

The above definition concerns surfaces with opposite curvative and in particular hyperbolic paraboloid.

A large structure, built for experimental work, has been under test for more than a year, and valuable information obtained for design purposes. It consists of a warped sheet of reinforced concrete about 2 in. thick, free of ribs or beams, 12.5 meters wide and 32 meters long, cantilevering 13 meters on both sides of the two supporting walls 6 meters long located centrally along the long sides of the structure. The cross section of the sheet at the supporting walls in a curve of considerable rise gradually flattening to a straight line at the ends of the cantilevers.

In conclusion, it is said that one of the important uses of monolithic reinforced concrete, because of its continuity and multi-directional resistance, lies in its application to thin shells, curved in one or more directions for the construction of roofs of large spans.

Construction of bridge on the Loire at St. Thibault made of vibrated fused (high alumina) cement concrete

SOUILLOT, *Journal L'Entreprise Francaise*, July, 1934, p. 4. Reviewed by P. H. BATES.

This bridge of 5 main arches is about 398 meters long with approaches about 406 meters. Due to the existing conditions, the arches are very flat and "differ very little from a parabolic curve." Further, conditions lead to the use of vibrated fused cement concrete—"thanks to the high compression of 'ciment fondu', one can use sections relatively reduced" . . . "vibrated 'ciment fondu' permitted lowering the dead weight on the floor" . . . "the direct consequence would be the diminution of the quantities placed in the work, the weight of which would compensate for the higher cost of the materials." High strength steel was also used and the author states, "for improving the use of reinforced concrete, whose weight, its principal enemy, has always been a difficulty in the construction of extended work, it is certain that a modern slogan would be 'for high strength reinforced concrete use high strength steels.' "

Aware of the inferior quality of the high alumina cement concrete resulting from the heat generated when cast in large sections, the ribs of the arches were cast in two longitudinal parts, using as a form a mesh on the side which would later be the center of the rib. After casting the first half, it was copiously sprinkled with water for 18 to 22 hours, and the second half cast directly against the side where the mesh had been used as a form. Excellent bond between the two halves was secured.

Tests of the concrete were made before the work was started and the work and sections were designed upon the obtained strength. Static and rolling load tests were made after the completion of the bridge, the deflections were well within the maximum expected limits, and after removal of the load no appreciable deflections or cracks could be noted. There is also described, too briefly, the strain gages used in the tests. They were of the glass mirror type inserted in small concrete blocks, which in turn were placed in the structure. Polarized light was reflected through the mirrors and the resulting tint noted and compared with that produced in an auxiliary mirror placed under a definite load.

Temperature rise in hydrating concrete

N. DAVEY and E. N. FOX, *Department of Scientific and Industrial Research, Technical Paper No. 15* 1933. Reviewed by N. M. NEWMARK.

There is an appreciable rise in temperature in cement mortar and concrete during the process of setting and hardening due to the exothermal nature of the chemical reactions between the water and the cement. In large masses of concrete the rise in temperature may be considerable, and this rise may not only affect the properties of the material itself, but may also influence the distribution and intensity of the stresses throughout the mass.

In this paper there is given an analysis of the problem, and a laboratory method for measuring the temperature rise under conditions which it is claimed simulate those encountered in practice.

The amount of heat involved by a cement mortar or concrete during setting depends among other things upon:

(a) Type and quantity of cement
(b) Size and shape of the mass
(c) Nature of the radiating surface of the mass
(d) Quantity of water used
(e) Degree of aeration of the cement

By curing the sample of concrete adiabatically it is possible to simulate very closely the conditions at the center of a large concrete mass, and so to estimate with fair accuracy the rise in temperature likely to occur and the amount of heat involved. In such a method very small samples can be used, as the results are independent of the size of sample tested; moreover, the variation in velocity of reaction at different temperatures is automatically taken into account. The method was first described in October, 1931, by N. Davey, "Temperature Rise in Concrete," Concr. Constr. Eng., 1931, Vol. 26, No.10 p. 572-5.

Several series of tests carried out with the adiabatic method of curing are described. A brief account is included in the paper of a mathematical theory derived by the authors. In the appendix, particulars are given of observations on a number of structures—dams, bridges, walls, slabs, etc., erected in different parts of the world.

Thin self-carrying shell vaults of reinforced concrete

HORMIDAS and R. VALETTE, *Le Genie Civil*, Vol. No. 104, No. 16, p. 355 to 359. Reviewed by R. L BERTIN

The article by Mr. Hormidas is in answer to the one by R. Valette, published in *Le Genie Civil* of Jan. 27, 1934, a review of which appeared in these pages.*

The author compares the method of design given by Mr. Valette with the one relating to the membrane theory of thin shells established by Dischinger (*Le Genie*

*Mar. Apr. 1934, *News Letter* p. 18; *Proceedings*, Vol. 30, p. 541.

Civil, June 9, 1928 and Feb. 2, 1929, also "Handbuch fuer Eisenbetonban," Vol. 6, Ed. 4, p. 271) and the more general one developed by Dr. Finsterwalder (see The International Association for Bridge and Structural Engineering of Zurich, Vol. 1, 1932), which has been used extensively for the design of a variety of roof structures successfully erected in various parts of the world.

The author points out that according to the membrane theory, in thin shell vaults of cylindrical cross section closed at their supports with transverse rigid diaphragms, when the tangent to the directrix at the edges of the vault is vertical and its edges unrestrained, the loads are transferred to the supports by the shell and supporting beams at the edges are not required.

This construction, however, requires vaults of considerable rise with reference to the spread of the vault and, in practice, it is desirable to terminate them short of the point where the tangent is vertical. This condition, however, vitiates the statically determinate solution of the vault described above, and with the introduction of edge beams and adjoining vaults acting monolithically, complex stress distributions are created, the determination of which is accurately solved by the method developed by Dr. Finsterwalder. It is noted that this solution is particularly accurate, as demonstrated by actual tests on models, because it is developed independently of any hypothesis relative to the stress distribution (the law of Navier) and takes into account the co-action of the shells and edge beams, conditions which are not considered in the solution offered by Mr. Valette.

In his closure, Mr. Valette states that his remarks were directed against the theory of Dischinger, the only one made public in France (*Le Genie Civil*, June 9, 1928 and September 2, 1929) which does not take the transverse moments into account.

He justifies his preference for retaining the law of Navier in his analysis by stating that the errors resulting therefrom in the computed lever arm and deformations are small and that it enables the easy handling of any loading for vaults of various shapes with or without edge beams.

Mortar tests as a guide to the strength of concrete

W. H. Glanville and D. A. G. Reid, *The Structural Engineer*, Vol. 12 (New Series), No. 5, May, 1934 p. 242. Reviewed by N. M. Newmark.

The paper describes a series of experiments carried out with the object of finding a small scale mortar test that may be used as an index to the strength of concrete made from the same cement.

The earlier tests were made on graded sand mortars in the proportions of 1:3 and 1:2 by weight, as well as standard (British) sand 1:3 mortars. The authors draw the conclusion that standard sand mortar strengths formed as good a guide to concrete strength as those with graded sand. Subsequent tests were made, therefore, on 1:3 standard sand mortars only.

Tensile, transverse and crushing tests were made on the mortars, tensile tests on the neat cement, and transverse and crushing tests on 1:2:4 concrete. For each form of test, three water contents were used representing dry, medium, and wet consistencies.

Comparison with mortar strengths are shown with one consistency of concrete only. The authors state that the strengths of the concrete of the different consistencies are definitely related. The strengths of the mortar of different consistencies are not similarly related; as a general rule the strength of the wet mortar is a greater proportion of the strength of the dry mortar for slow than for rapid-hardening cements.

The following conclusions are taken from the authors' summary:

The value of the mortar strengths as a guide to the concrete strengths increases as the consistency of the mortar is made wetter.

Neat cement and dry mortar mixes form a very poor guide to the strength of concrete, particularly when the various ages of testing are not considered separately.

The best guide to the crushing strength of concrete is given by the crushing strength of mixes of 1:3 standard sand mortar as used under the B.S.S. for portland cement, but with a water content of 12½ per cent of water by weight. The results enable the concrete strength at the same age to be estimated in the majority of cases, to within 10 per cent, and, in the worst case recorded, within 20 per cent. The coefficient by which the mortar strength has to be multiplied to obtain the strength of 1:2:4 (by weight) concrete, with a water-cement ratio of 0.60, has a mean value, for all ages and cements, of approximately unity.

The best guide to the transverse strength of concrete is given by the transverse strength of wet mixes of 1:3 standard sand mortar, although the tensile strength of these mixes is practically as good. The percentage range is about the same as already indicated for crushing strengths.

A few tests made with high alumina cements have shown that the wet mortar strength may be used as guide to concrete crushing strength within the same degree of accuracy as may be expected with portland cements. The mean ratio of the coefficient by which the mortar strength must be multiplied again approximating to unity.

Relation between the characteristics of concrete and the use of vibration

J. LEBELLE, *Le Genie Civil*, Vol. 104, No. 15, p. 336 to 338. Reviewed by R. L. BERTIN.

Reviews the application under various conditions of mechanical vibration for the compaction of concrete which is supplanting more and more the manual operation because of qualitative improvements.

The most essential advantage of compaction by mechanical vibration is that it makes possible the use of much drier concretes, resulting in greater strength and density and reduced shrinkage, particularly if the proportions of finer materials, sand and cement, are reduced to a minimum consistent with quality of concrete desired.

The "floating pervibrator," consists of a vibrating unit enclosed in a tight metal shell so proportioned that when submerged it displaces a volume of concrete of greater weight than itself. Hence, when placed in a deep form and concrete deposited gradually on top of it, the unit rises automatically as it liquifies the mortar. The claim is made that it serves as a means of virtually controlling the quality of the concrete, since the material in excess, either coarse aggregate or mortar, rises to the surface and corrective measures may be taken as the work progresses.

Because of greater uniformity in vibrated concrete a reduction of factor of safety is suggested.

The various methods of vibrating concrete are grouped as follows:

(1) External vibration—vibration transmitted to the concrete by vibrating elements attached to the forms.

(2) Superficial vibration—transmitted to the top face of the concrete by vibrating platforms.

(3) Internal vibration—by the immersion of vibrating units.

(4) Semi-superficial vibration—by vibrating platforms with projecting members which penetrate the mass.

External vibration is effective only if one of the dimensions of the piece is small, otherwise only the concrete immediately in contact with the forms is benefited. It requires particularly strong and tight forms.

Superficial vibration penetrates to a depth varying with the consistency and is suitable for large surfaces and large masses; the thickness of successive layers must be controlled to prevent segregation in layers. This tendency is greatly reduced by the use of the semi-superficial vibrating tools.

Internal vibration, particularly with the floating pervibrators, is considered the most effective. However, it is limited to deep sections such as walls, columns and the like.

Where impermeability is of prime importance, a mortar containing more fine elements is recommended which may be sand dust if strength is not important, and lime or cement in such proportions as to maintain the cement-water relation constant when both strength and impermeability control.

Where the floating pervibrator cannot be used, the blade type of internal vibrator used singly or in conjunction with the platform vibrator gives good results, although the resulting compaction is not as great. For reinforced structures, more fluid mixtures, using smaller aggregate and larger proportions of mortar, drier however than required for manual placing, can be handled successfully with the blade pervibrators, resulting in concrete which is more uniform and less subject to shrinkage.

Some properties of cement and concrete

Abstract of a paper before the Society of Swedish Engineers in Great Britain, by D. Werner and S. Giertz-Hedstrom. *Cement and Cement Manufacture*, Vol. 7, No. 6, July, 1934, p. 211-28. Reviewed by J. C. Pearson

The paper presents some of the results of a general investigation of present-day cements at the Royal Swedish Institute for Engineering Research, where incidentally, discriminating attention has evidently been given to the reasonableness of demands for cements with special properties. The authors point out that some of these properties must be secured at a sacrifice of other desirable properties, and that the requirements of economical production restrict the choice of cements to a limited number of types.

A study of strength was made by 3 methods to determine which was most indicative of the performance of different cements in practice. These methods were: the present Swedish Specification, including tests of 1:3 dry mortar cubes, machine tamped; the new Swiss Specification prescribing transverse tests of hand-tamped 1:3 mortar prisms 4x4x16 cm., and compression tests on the prism pieces resulting from the bending tests; and a method proposed by Dr. Hägermann, similar to the Swiss method, but involving a more plastic, graded sand mortar. The Hägermann method was found much superior to the others, giving better relations between transverse and tensile strength, and strengths more nearly on a par with those secured in practice. Quite remarkable is the confirmation of Feret's law, found by this method to hold for all types of portland cement, and given by the equation

$$T = k\,C^{3/4} \qquad (1)$$

The authors state that it is now superfluous to measure both tensile and compressive strength, as one can be calculated from the other, the transverse tensile strength being the simplest and most accurate to use. For neat cement the following simple

relation between strength properties was found, presumed to hold good also for mortars:

$$E = K \sqrt{C \cdot T} \quad (2)$$

An extensive study of the durability of concrete pipes in aggressive waters is reported briefly, in which it was found that humus was not an aggressive agent in itself but was usually associated with soft waters containing dissolved CO_2. Degree of acidity of a water (pH) does not give a simple measure of aggressive action, as this also depends upon the amount of active CO_2 available. The best all-around protective coating for pipes was a priming treatment of asphalt dissolved in naphtha or some other suitable solvent, followed by a coating of warm asphalt.

In relation to volume changes and other properties, a study of the water fixation in set cement was made. It was found convenient to consider this in three different classes: (1) fixed water, chemically combined; (2) half-fixed water, that retained in the gel; and (3) free water, enclosed in the coarser pores of the material. Fixed water is obviously related to strength, and formulae have been developed giving the relation between strength and the volume percentage of the solid phase. Half-fixed water is related to volume change, the loss of this water setting up high capillary forces which compress the material, and this is manifested outwardly as a shrinkage. The same forces are also the cause of the increase in mechanical strength observed in specimens of cement and concrete when dried. If free water is defined as that which can be dried out of the specimen by means of air at 97 per cent R. H., this implies that free water occurs in pores which are greater than .00007 mm. in diameter. Since the free water is thus a measure of coarse porosity of the material, it determines the permeability to water at low pressure. At high pressures, permeability may depend also upon the water in the very fine pores—a part of the half-fixed water.

Risk of cracking is uniquely discussed, and is defined as

$$R = \frac{\sigma}{T} \text{ or } \frac{\Sigma \cdot E}{T}$$

where σ = tensile stress, T = tensile strength and Σ = unit deformation. If R is less than 1, the material will not crack, but the condition is critical when $R = 1$. For this condition, $\Sigma = \frac{T}{E}$, and from the relations given in equations (1) and (2) the critical value is

$$\Sigma = \frac{k_1}{T^{1/4}} = \frac{k_2}{E^{1/5}} = \frac{k_3}{C^{1/6}},$$

the interpretation of which is that the material may show the greatest contraction when its strength is lowest.

Temperature differences which are critical as regards the risk of cracks can be calculated from the formula

$$\Delta t = \frac{\Sigma}{a} = \frac{T}{aE}$$

in which Δt is the temperature difference and a is the thermal coefficient of linear expansion. Test values are given for a normal cement of good quality from which it appears that the critical temperature difference is about 25° C., and apparently independent of water content and age. Reference is made to a special low-heat cement used during the past two years in Sweden, comparable to the type specified for Boulder Dam, but greatly exceeding the minimum requirements of the latter in strength.

Cements unaffected by absolutely pure water

P. Dumolard, *Le Genie Civil*, Vol. 104, No. 26, June 30, 1934, p. 584. Reviwed by R. L. Bertin

Presents in historical review observations regarding the disruptive effect of very pure waters on mortars and concretes, and indicates how to guard against such attack by judicious selection of the cements.

A concrete aquaduct built for the City of Vannes in 1917 which after 10 months of service had disintegrated, marked the beginning in France and elsewhere of much research, the results of which are given in this article in considerable detail. The damage consisted at first of a dark crown on the interior, edged at first by a gelatinuous zone gradually turning white. Similar zones were noticed on the outside under damp exposure. In these damaged sections, the concrete was softened sufficiently to fall apart under the pressure of a finger.

The investigators of this particular case concluded that for aquaducts conducting pure water, artificial cements should not be used and should be constructed with prompt cements or cast iron. Prior to this investigation, Mr. Mesnager had indicated that cast iron as well as artificial cements are attacked by pure waters.

The first part of the article deals with the attacks of cements and mortars by distilled water. It cites the work of Candlot "Cements and Hydraulic Limes" in which the loss of lime in distilled water for various cements is given, the work of Mr. Giran in 1931 on solubility of ground set cement, also that of Werner, and the results of similar research by Hurgerman, Rodt, Brehl, Sundino and Assarson, Virgin, contained in a paper by Roos, entitled "Action of Aggressive Waters on Cements," presented at the Congress of Zurich held in September 1931.

The conclusions of this paper are as follows:

In the presence of pure water or water containing bicarbonate of lime, or carbonic acid, the different artificial cements possess approximately the same solubility. The high alumina cements yield lime and alumina and the total effect of dissolution is as important as for the artificial cements.

The superficial carbonation in damp air increases sensibly the resistance to the attack of water especially if it is free of carbonic acid. Certain acid substances or pozzuolan of silica or silicate base reduce the solubility of the lime.

The second part deals with the action of water on the silicates and aluminates of lime.

It cites the work of A. LeChatelier entitled "Experimental Research on the Constitution of Hydraulic Mortars," also that of Lafuma entitled "Research on the Aluminates of Calcium", both of which show that all silicates and aluminates of lime are attacked by distilled water, and which give the limits of decomposition by hydrolysis.

The author points out, however, that the limits of hydrolysis in the case of cements exposed to running water are not of as much importance as the speed of reaction. Additional tests were made in which the amounts of water were varied. The results of these tests are confirmed in the later work of Dubrisky and Lafuma, entitled "Study of the Volume Increase which Accompanies the Hydration of Cements by A Large Excess of Water," also that of J. C. Witt and F. D. Reyes who found that an artificial cement in the presence of a large excess of water can lose 87 per cent to 92 per cent of its lime.

The author draws the conclusion that the speed of hydrolysis in the presence of large excess of water is very slow for prompt cements and more rapid for artificial

cements. As to high alumina cements, they seem to be attacked faster than the artificial cements.

The third part deals with the accessory phenomena which can contribute to the resistance of cement to pure water.

Impermeability is given as the most important and then again prompt cements are advantageous for the following reasons:

In high silica cements (artificial cements) only a small percentage of the water used in mixing combines with the silica, the volume of the free water resulting in voids, while with prompt cements containing aluminates or sulfoaluminates of lime crystalizing with 12 or 30 molecules of water, the entire water is absorbed chemically thus producing higher impermeability.

The author refers to additional early work by M. E. Maynard and later work by Lafuma presented in September 1933 at the Congress of Chemistry at Lille which deals with the carbonation of lime in cements. He brings out that if the solution contains less than half a gram of lime per litre, the carbonate is adherent which is the case with prompt cement, whereas for higher concentrations, the precipitate is amorphous. In the first case the carbonate settles in a solid insoluble mass; in the second case, in the presence of running water, it is washed out.

Finally, the conclusions of Colonel Mourral (*Le Genie Civil*, Feb. 4, 1928, p. 121) are quoted as follows:

Prompt cements, already protected by their impermeability to the action of pure waters do not contain any free lime, a substance particularly sensitive to these waters, and it follows that they allow but very small quantities of lime originating exclusively from hydrolysis to pass in solution; a most satisfactory condition for the formation of an adhering carbonate remaining within the mass, a contrary condition occurring with artificial cements.

The author closes with a statement that the prompt cements therefore possess all the properties conducive to resisting the action of pure water as demonstrated not only by many laboratory tests but known by experience for nearly a century, and further demonstrated by the good results obtained with a coating of prompt cement to protect certain dams against the attack of pure water.

Consistency Indicator for a Ready-Mixed Concrete Plant*

BY E. B. RAYBURN, JR.†
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To provide and maintain a constant consistency is one of the chief problems of concrete production. This is doubly true of a ready mixed concrete plant, with a variety of mixes and consistencies to be supplied in any one day's run. Failure to provide a uniform product upsets placing operations, causes a variation in strength, increases costs for both the customer and the producer because of delays and creates a bad impression of the product that is hard to overcome.

In attempting to control the consistency, visual inspection has generally been resorted to in the absence of a better method. Obviously this cannot be very satisfactory as it is largely a matter of guesswork, with the best guesser producing the most uniform concrete. At the same time it is impractical and expensive to attempt to use visual methods for controlling consistency in a ready mixed concrete plant.

Some mechanical devices for indicating consistency have been developed within the last few years (one as early as 1928) which eliminate the personal element and provide a more accurate measure of the slump. These devices have assumed divers forms and shapes since both their design and application reflect their author's philosophy of concrete mixes. One of these devices was described in *Pit & Quarry* for December 1931, another in the *Proceedings of the A. S. T. M.* for 1930 and another in the Journal of the American Concrete Institute for September 1931.[1] Another such apparatus which has been in successful operation since February 1934 is described herein.

TYPE OF INSTALLATION

The device is functioning in a central mixing plant where mixing is done in a 2-cubic yard stationary mixer. Four cubic yard agitators are used for delivery, requiring two batches to the load. Materials

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[1] "Determining Characteristics of Concrete in the Mixer Drum," by Emory D. Roberts, *Proceedings* Vol. 28, p. 59.

are batched by weight, using bulk cement, two sizes of coarse aggregate and one of sand. Annual production for the last two years has averaged about 35,000 cu. yds. Approximately 50 per cent of this yardage is used in paving work, concrete being sold on a strength specification with a penalty attached if the strength falls below a specified minimum.

There is a total of some forty odd concrete mixes, ranging in consistency from slumps of 1 in. to 8 in., and in strength from 1500 p.s.i. to 6000 p.s.i. All sales are made on the basis of a specified or recommended strength and consistency, with the strength being constantly checked by both the customer and the company.

Mixes are designed by trial in the laboratory. It has been found that the constant water content theory advanced by Inge Lyse[2] works very well for mixes in the range of 2000 p.s.i. to 4000 p.s.i. Above 4000 p.s.i. there must be a slight correction due to the thickening of the paste, and below 2000 p.s.i., there must also be a correction because the reverse takes place. The percentage of sand to coarse aggregate is varied according to the richness and consistency of the mix. Thus lean mixes contain a higher percentage of sand than rich mixes and a wet mix contains more sand than a dry one. This variation has nothing to do with any "optimum" content but is based on the observation of many batches of concrete produced in the field.

Control is predicated on a theory which at first blush may appear to be putting the cart before the horse. Anyone who has seen concrete produced in large quantities knows that it is impossible to obtain an absolutely uniform moisture content in the aggregates, and that most of the time it would require continual checking to know just what that moisture content was. For this reason batches are designed for a definite w/c ratio and consistency in the laboratory, and this consistency is then produced in the mixer. It is assumed (and borne out by the test results) that the total water content, as determined in the laboratory is therefore correct, if all other factors remain the same. The use of split sizes of coarse aggregate allows enough flexibility to maintain a practically constant grading, and if the sand grading is approximately constant, the producer is safe to assume that the foregoing premise is correct. There remains then the problem of maintaining a constant consistency, and when this is accomplished, the producer will be supplying concrete of uniform strength. The apparatus about to be described has effectively solved the problem for one ready mixed plant.

[2] "Tests on Consistency and Strength of Concrete having Constant Water Content," *Proc. A. S. T. M.*, Vol. 32, 1932, Part II, p. 629.

THE APPARATUS

Any device for controlling consistency should satisfy the following requirements:

1. Simple and positive action
2. Few working parts
3. A minimum of parts subjected to wear in the mixer and all working parts outside the mixer
4. Sensitivity to a slight change in water content
5. Positive indication of the consistency of the concrete whether very wet or very dry.
6. Durability.

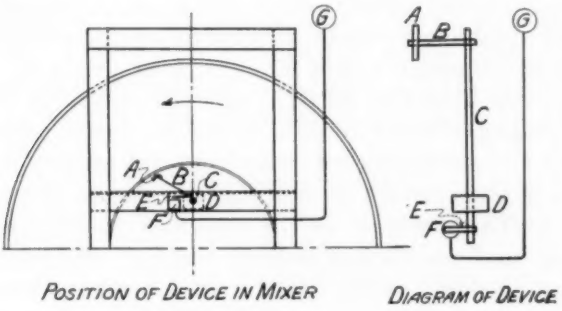

FIG. 1

The scheme shown in Fig. 1 was selected as best meeting the requirements. It has been simplified to the point of crudeness, but the absence of any major repairs during its service has proved the wisdom of this simplicity. No dimensions are given since these will vary with the type of installation and the range of readings desired.

The direction of rotation of the mixer is shown by the arrow in Fig. 1. Concrete falling from the blades strikes (A) and (B) and tends to rotate the shaft in a counter-clockwise direction. The shaft (C) is given freedom of motion by the bearing (D) and would revolve if not prevented by the arm (E) which rests on the oil bellows (F). This bellows is connected to gauge (G) which indicates the resistance in p.s.i. necessary to prevent rotation of the shaft (C). Obviously, the stiffer the concrete, the more resistance required and vice versa.

This scheme affords a maximum of simplicity and requires no working parts to be exposed to the erosive action of the concrete in the mixer. The few exposed parts can be cheaply and easily replaced should they be worn out. It will be noted that the indicating arrange-

ment is much the same as that described by Emory D. Roberts in his paper in this JOURNAL for September 1931. This was used because of the ease of obtaining a steady reading on the gauge. If the line from the bellows to the gage is left entirely open, the readings will fluctuate over a range of 5 to 10 p.s.i. due to the intermittent action of the concrete falling from the mixer blades, but if the line is restricted by partly closing a shut-off cock the pulsations are so eliminated that a steading reading is obtained, which is in a direct ratio to the consistency of the concrete. A fluid was used in the line which was not influenced by temperature changes, thus eliminating any correction on that score.

APPARATUS IN OPERATION

As previously noted, approximately 50 per cent of the plant yardage is composed of paving concrete, of medium to dry consistency, using coarse aggregates graded to 1½ in. The recording gage was graduated in intervals from 1 to 30 p.s.i., and the apparatus was adjusted to show a reading of about 15 p.s.i. for a 4-in. slump. This permitted ample range and sensitivity for either very wet or very dry concrete.

Preliminary observations showed that the pressures were in direct proportion to the consistency of the concrete. It was possible to dry the batch up until a reading of well over 30 p.s.i. was reached and the concrete had no slump at all. On the other hand the batch could be made wet enough to show a reading of only 3 p.s.i., where the concrete might properly be called "soup."

The calibration of the gage for the different slumps given below is for 1½ in. aggregate, and is typical for the ¾ in. and mortar mixes.

Slump—Inches	Gage Reading
1	25.0
2	21.5
3	18.0
4	15.0
5	12.5
6	10.0
7	8.0
8	6.0

The results from the use of the device were very satisfactory from the beginning, and much more so than the old visual method at its best. After a few weeks several things were noted about its performance which were interesting and which had to be reckoned with if a constant slump was to be maintained. Some of the things listed merely substantiate the observations of others, but several of them seem peculiar to this apparatus.

1. The gage reading became constant in from 30 to 45 seconds after the materials were in the mixer, depending on the richness and consistency of the mix.

2. As the batch was introduced into the mixer the pressure rose slowly, dropped back slightly, then rose to a maximum which was maintained until the batch was discharged. Another method of charging might cause the pressure to rise to a maximum then drop back slowly to a constant and lower reading.

3. Batches mixed up to five minutes showed no appreciable change in pressure or slump. After five minutes the pressure rose slowly, requiring additional water to restore the original pressure.

4. The addition of one half gallon of water per cubic yard was sufficient to decrease the gage reading from ½ to 1 p.s.i. depending on the consistency of the concrete at the time of the addition of water.

5. Cutting the batch in half—from 2 cubic yards to 1 cubic yard—decreased the pressure for a given consistency about 1 p.s.i.

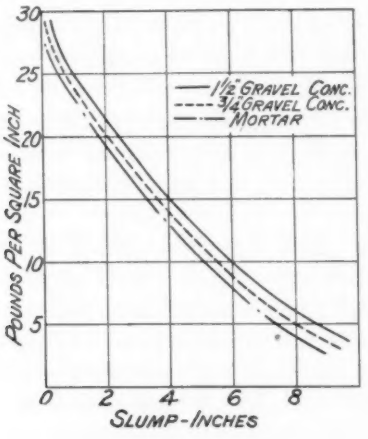

FIG. 2

6. When aggregates graded up to ¾ in. were used instead of 1½ in. aggregates, the reading for a given consistency was decreased about 2 p.s.i. In mortar mixes this reading was decreased an additional 2 p.s.i., or 4 p.s.i. less than for 1½ in. aggregates. Fig. 2 shows this relation over the ordinary range of consistency.

7. With any given pressure, the consistency as measured by the slump cone was slightly affected by the richness of the mix and was in direct proportion to it. Example: With a pressure of 15 p.s.i., the

slump for 5000 lb. concrete was 5 in., but for 2000 lb. concrete, the slump was only 3 in.

8. For a given pressure, the consistency varied according to the grading of either the coarse aggregates or the sand, the slump increasing slightly as the percentage of the coarser sizes was increased.

9. No appreciable change in the total water content was occasioned by the variations noted in 8. This of course is not true where the sand varied from very fine to very coarse or vice versa, while the proportion of fine to coarse aggregate remained unchanged.

10. For a given pressure, different cements showed a slight variation in slump, the finer cements showing the most slump.

DISCUSSION OF RESULTS

While it was not possible to use laboratory methods in arriving at the results, great care was exercised to insure accuracy. Fortunately the plant was producing a large and varied yardage during the time of observation. Cement contents of the mixes ranged from 3½ to 7 bags per cubic yard, and the concrete was used for paving, foundations, thin wall sections, reinforced sections of all kinds and floor topping. Batches were observed for their behavior on being discharged from the mixer into the truck, from the truck to the subgrade or receiving hopper, etc. On the job the batches were observed under the various placing operations—screeding and finishing with a machine, puddling with sticks, and/or vibrators in reinforced sections, and handling with shovels on the grade.

The paving work offers a good example of the effect caused by changing cement content. Two sections of pavement were placed, each comprising about 8,000 cubic yards of concrete. The same cement was used for practically all of one section and part of the other. The specified slump was the same for each section—2 to 3 in.—but the cement contents were 1.7 and 1.5 barrels respectively. Some trouble was experienced in maintaining a 3 in. slump for the richer mix, as the cement content was so high that the mass had very sticky characteristics: it piled up in the trucks, was slow flowing down the chutes, clung to the shovels of the men spreading it on the grade, and "drug" on the finishing machine. When the second section was begun the same crew and foremen were on the job. They immediately remarked that we (the producer) had at last learned how to make concrete. It handled easier for all concerned, and the inspector, noticing its wetter appearance, made numerous slump tests, but they were all 3 in. or less. Now the reading on the gage for a 3 in. slump, using 1.7 barrels per cubic yard was about 21 p.s.i., but was only 18 p.s.i. for the con-

crete containing 1.5 bbl., definitely showing that the richer mix offered the greater resistance. On the other hand if the concrete had been produced with the same reading of say 18 p.s.i., the richer mix would have shown a slump of about 4 in. The same relation was noticed for batches of still lower cement contents, but as the mixes became leaner, the resistance due to the thinning of the cement paste and the consequent friction of the aggregates on each other tended to offset the decrease in resistance offered by the paste.

The effect of the richness of the mix was perhaps the most important of all. However, if different brands of cement were used, with appreciable variations in fineness, it was necesxary to take this into account if a constant slump was to be furnished. Four cements were used during the period of observation—one high early strength cement with a specific surface of approximately 2400, and three ordinary portland cements with specific surfaces of approximately 2000, 1600 and 1500. On the basis of equal pressures, the finer cement showed the higher slump. Conversely, using 1.7 barrels and a 3 in. slump, the high early strength cement was harder to handle than the ordinary portland with a specific surface of 2000, and so on down the line.

The effect due to increase in coarser sizes of gravel seems logical. While this increase would naturally reduce the voidage and/or surface area of the aggregate, with a consequent increase in the consistency of both the cement paste and the entire mass, this very increase in the consistency of the paste, without an increase in the mortar content, allowed more friction between the particles, so that the pressure exerted on the bar was practically the same as that of the mix having a more evenly graded aggregate but of a lower consistency.

It would appear that the apparatus measured both workability and consistency. A constant consistency could be easily supplied for any one mix, as long as the aggregates were properly controlled. A change of mix or cement would require a slightly different reading on the gage if the same consistency was desired, i.e., 4000 lb. concrete at 5 in. slump vs. 2000 lb. concrete at 5 in. slump, or the consistency could be disregarded and a constant workability supplied for all mixes. From the producer's point of view the choice would depend on how much any of the previously mentioned factors varied. If only one brand of cement was used and the range of mixes was slight—from 2000 to 3000 lb. say—then it would be easiest to produce a constant consistency, and at the same time the workability of all mixes for any one degree of consistency would be practically the same. At the same time, the producer could determine the best aggregate combinations and proportions for the different mixes, i.e., the "optimum" proportions.

CONCLUSION

The advantages of mechanical control are as follows:

1. It is practical and inexpensive.

2. More information about the character of the batch is provided at the producing end instead of waiting until the concrete is handled on the job before knowing whether it is suitable or not. The result is greater uniformity in consistency and strength.

3. It not only indicates consistency, but also shows the comparative workability of the concrete.

4. The greater uniformity provided allows a reduction in the factor of safety or "over design" of mixes, with a slight but welcome (from the producer's standpoint) decrease in cement content per batch.

5. The control engineer is given more freedom for research, contact with customers, etc., instead of being tied down to making moisture determinations and other routine tests. The actual production can be intrusted to an intelligent but not particularly educated mixer man, since the only information necessary to enable him to provide concrete of any strength and consistency is the mix quantities and the corresponding gage readings.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for May-June 1935. Discussion should be available to the Secretary by April 1, 1935.*

Flow of Heat in Dams*

BY ROBERT E. GLOVER†

INTRODUCTION

The abutments against which arched dams are constructed resist the expansion and contraction of the concrete which temperature changes would produce if the dam were unrestrained, with the result that the necessary adjustment of shape is accompanied by elastic distortions and stress changes. The important influence which temperature changes may exert upon the stress distribution in arched dams has long been recognized and it has become customary to allow for this factor when preparing designs for such structures. Temperature changes in thin dams result mainly from seasonal changes of temperature which influence the thermal state of massive dams to a limited extent only. The factors, however, which make massive dams largely immune to the effect of seasonal changes subject them to danger from another source. During the setting process cement liberates a certain quantity of heat which raises the temperature of the concrete mass above the temperature at which it was placed. In slender dams this heat of setting, together with any excess due to high placing temperatures, may be largely dissipated during the construction period but in large dams the flow of heat to the surface is so greatly retarded that an undesirable amount of heat may be retained at the time of grouting to be released later.

Heat retention is favored both by large dimensions and rapidity of construction and since both these factors were to be present at the Boulder dam to an unusual degree it was essential that accurate formulas be made available for estimating the magnitude of the temperature changes to be expected. This paper contains formulas and diagrams prepared for that purpose.

The paper deals primarily with the heat flow problems encountered in the design of slender arched dams, and the treatment is, therefore, confined to the calculation of the time required to lose the heat of

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†Engineer, U. S. Bureau of Reclamation

setting and the effects of external temperature changes. The mathematical basis of these formulas, as well as those developed for the calculation of heat retention and the design of cooling systems for massive dams, will be presented in a bulletin on the subject to be issued at an early date by the Bureau of Reclamation.

THE MATHEMATICAL THEORY OF HEAT CONDUCTION

The mathematical theory of heat conduction may be said to have begun its existence with a treatise published in the year 1822 by the French mathematician Fourier under the title "Theorie Analytique de la Chaleur." The methods employed by Fourier aroused considerable interest, and during the 112 years which have intervened the subject has been reviewed and amplified by many able mathematicians and physicists. The results of these labors may now be found in texts dealing with the subject,[1] to which the reader is referred for the detail of the mathematical processes.

The study of temperature changes requires the use of a characteristic number called the diffusion constant or the diffusivity. It is numerically equal to the conductivity divided by the product of the specific heat and density. If this number is large the body will go through the temperature change rapidly. If it is small the change proceeds slowly. The diffusion constant has the physical units length squared divided by time.

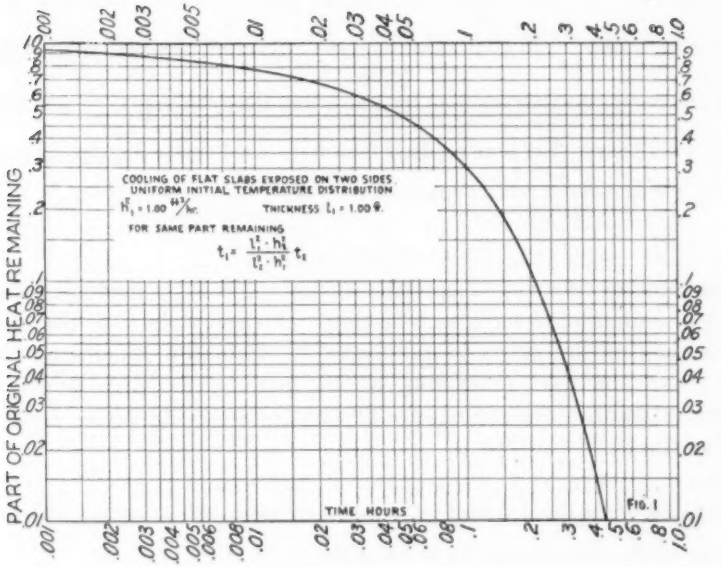
THE COOLING OF A FLAT SLAB

The study of the cooling of a flat slab is of interest because of the information which it gives regarding the time required to dissipate the heat generated in a dam by the cement during the setting process. The result of the solution of the heat equations for this case are contained in Fig. 1. The diagram gives directly the time required for the loss of a given proportion of the initial heat from a slab one foot thick and having a diffusion constant equal to one foot square per hour. Correlation to the actual case is made by means of the correlation formula:

$$\frac{h_1^2 t_1}{l_1^2} = \frac{h_2^2 t_2}{l_2^2} \qquad (1)$$

where the quantities h^2 t and l represent respectively the diffusion constant, the time and the thickness of the slab. The subscript "1" refers to the conditions represented on the chart, and subscript "2" to the case which is to be calculated. The manner of using this formula and the chart is shown in the following example:

[1]References: Carslaw, "The Conduction of Heat," McMillan & Co. Ingersoll and Zobel, "The Mathematical Theory of Heat Conduction," Ginn & Co.


COOLING OF FLAT SLABS EXPOSED ON TWO SIDES
UNIFORM INITIAL TEMPERATURE DISTRIBUTION
THICKNESS l_1 = 1.00 ft.
FOR SAME PART REMAINING
$t_1 = \frac{l_1^2 \cdot h_1^2}{l_2^2 \cdot h_2^2} t_2$
PART OF ORIGINAL HEAT REMAINING
TIME HOURS
FIG. 1

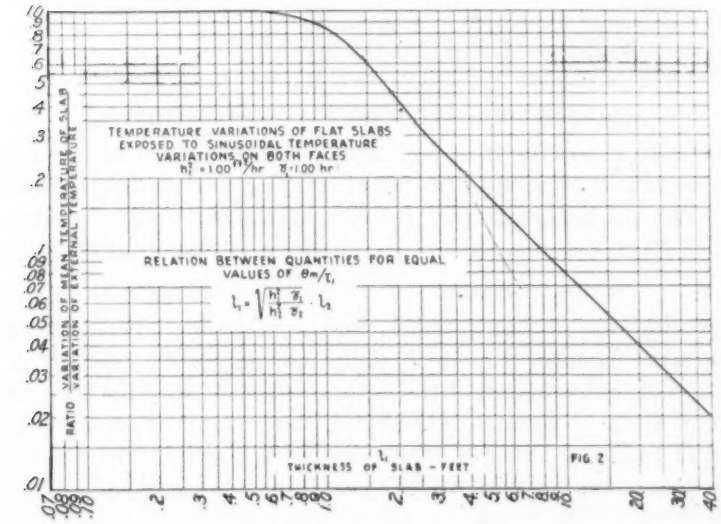

TEMPERATURE VARIATIONS OF FLAT SLABS
EXPOSED TO SINUSOIDAL TEMPERATURE
VARIATIONS ON BOTH FACES
RELATION BETWEEN QUANTITIES FOR EQUAL
VALUES OF $\theta m/t_1$
$l_1 = \sqrt{\frac{h_1^2}{h_2^2} \cdot \frac{\theta_1}{\theta_2}} \cdot l_2$
RATIO VARIATION OF MEAN TEMPERATURE OF SLAB / VARIATION OF EXTERNAL TEMPERATURE
THICKNESS OF SLAB - FEET
FIG. 2

FIG. 1 AND 2

Example 1.

At a certain elevation a dam is 50 ft. thick. If it is exposed to the air on both sides how long will be required for 90 per cent of the heat of setting to be lost?

The dam will be considered as a slab of the same thickness. Assume h^2 for this concrete to be $.040 \frac{ft^2}{hr}$. If 90 per cent has been lost then 10 per cent or 0.10 remains. From the curve, the time required for a slab 1 ft. thick, $h^2 = 1.00 \frac{ft^2}{hr}$ to lose 90 per cent is 0.212 hours. The time required for a 50 ft. thickness is by formula (1).

$$t_2 = \frac{1.00}{0.040} \cdot \frac{50^2}{1.00^2} \cdot 0.212 = \begin{matrix} 13250. \text{ hours.} \\ \text{or } 1.51 \text{ years.} \end{matrix}$$

COOLING OF A PRISM

The cooling of a prism can also be calculated by means of Fig. 1. The parts remaining for a slab having a thickness equal to each of the dimensions of the prism respectively is calculated by means of the chart, and the part remaining is obtained by multiplying the two quantities so obtained together. The procedure is illustrated by means of example two.

Example 2.

What part of the original heat will remain in a prism of rectangular cross-section 50 ft. by 100 ft. after a lapse of 1.51 years if $h^2 = 0.040 \frac{ft^2}{hr}$?

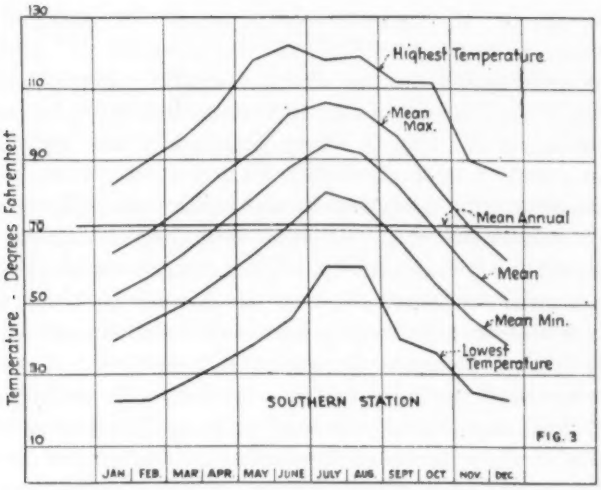
Part remaining, $l = 50$ ft., $h^2 = .040$, is 0.10.
Part remaining, $b = 100'$, $h^2 = .040$, is 0.48.
Part remaining in prism = (0.10) (0.48) = 0.048.

COOLING OF A RECTANGULAR PARALLELOPIPED

The part remaining in a rectangular parallelopiped exposed on all six faces can be obtained in a manner similar to that described above, by finding the part remaining for each of the three dimensions of the parallelopiped and multiplying the three parts remaining together.

SLABS EXPOSED TO VARIABLE TEMPERATURES

The solutions of this problem find application in estimating the temperature changes to which dams are subjected because of the variation of temperatures of the air and water with which the surfaces of the dam are in contact. As before, the problem is idealized to the extent of assuming that at each elevation the change which takes place in the dam is the same as would occur in a slab of the same thickness and material. The results of the calculations for this case are shown on Fig. 2. The diagram shows the variation of the mean temperatures of the slab in terms of the variation of the external temperatures for slabs of different thickness, having a diffusion constant equal to one foot square per hour, when subjected to a sinusoidal temperature



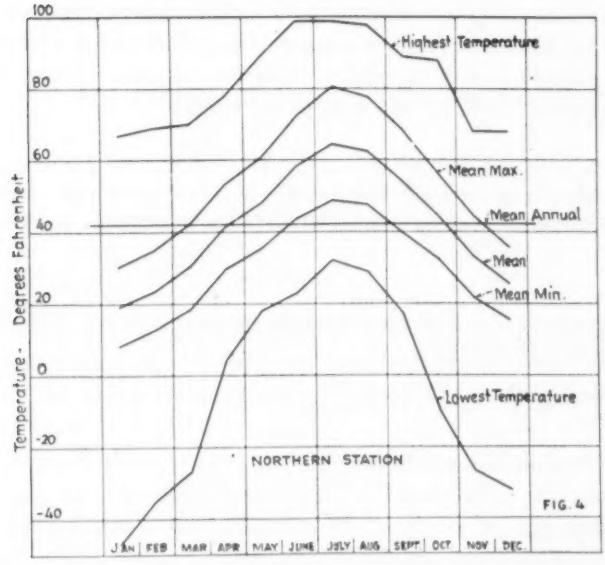


FIG. 3 AND 4

change with a period of one hour. As before, correlation to the actual case is made through the use of a correlation equation. This equation is:

$$\frac{h_1^2 \gamma_1}{l_1^2} = \frac{h_2^2 \gamma_2}{l_2^2} \quad \ldots\ldots\ldots\

where the symbol "γ" represents the period of the imposed temperature change. As in the previous case the subscript "1" applies to the conditions represented on the chart, and the subscript "2" to the actual conditions. The use of this diagram is illustrated by two applications in example 3. Fig. 3 shows graphically the weather bureau records for a station in southwestern United States, while Fig. 4 shows similar conditions for a station in the northwest. There are shown mean daily temperatures, the mean maximum and the mean minimum temperatures, the lowest and the highest temperatures observed, and the mean annual temperature. For the southern station the lowest temperature observed is about as far below the mean annual temperature as the highest temperature observed is above it. Also, the daily amplitude is about the same in the winter months as in the summer months. These conditions permit of sufficiently accurate expression by the sum of three terms having different amplitudes and periods. The conditions at the northern station are slightly more complicated. The lowest temperature observed is farther below the mean annual temperature than the highest temperature observed is above, and the daily amplitude is less in winter than in summer. These conditions can be approximated by the sum of six terms.

Example 3.

What variations in the mean temperature of concrete are to be expected if the surfaces are exposed to the air temperatures plotted on figures 3 and 4 if $h^2 = 0.050$ $\frac{ft.^2}{hour}$?

The temperatures shown on Fig. 3 were represented by the sum of the three following sine waves of temperature.

Annual	$-22 \cos \frac{2\pi}{8760} t$
Fifteen day	$-16 \cos \frac{2\pi}{365} t$
Daily	$-12 \cos \frac{2\pi}{24} t$

The temperatures of Fig. 4 were represented by the sum of the following six terms:

Annual	$-22.7 \cos \frac{2\pi}{8760} t$
Fifteen day	$-20.0 \cos \frac{2\pi}{365} t$
7 day	$-18.0 \cos \frac{2\pi}{182.5} t$

$$\text{7 day} \qquad -18.0 \cos \frac{2\pi}{178.776} t$$

$$\text{Daily} \qquad -12.5 \cos \frac{2\pi}{24} t$$

$$\text{Daily} \qquad -2.5 \cos \frac{2\pi}{23.9344} t$$

In both cases time is counted from January 15. The amplitudes and periods were selected to represent the characteristics of the temperature at the stations as nearly as possible with a limited number of terms. The yearly and daily amplitudes were first accounted for and the remaining term or terms selected after a study of maximum and minimum temperatures and frequency of temperature fluctuation. The amplitudes were chosen to represent maximum rather than average conditions. The results of the analysis are shown on Fig. 5. The individual computations were made in the following manner: The annual variation at the southern station has an amplitude of 22°F. and a period of 8760 hours. If the thickness to be considered be taken as 20 ft., then $\gamma_2 = 8760$ hr., $l_2 = 20$ ft., and from the statement of the problem $h_2^2 = 0.050 \frac{ft^2}{hr}$. The corresponding data for the diagram is $\gamma_1 = 1.00$ hr., l_1 unknown $h^2_1 = 1.00 \frac{ft^2}{hr}$. If these values are substituted in the correlation formula, there results:

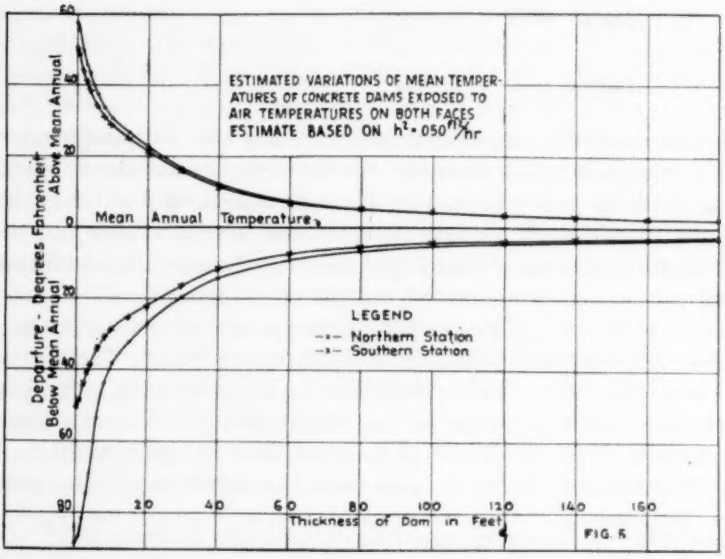
$$l_1 = \sqrt[2]{\frac{(20^2)\ (1.00)}{(0.050)\ (8760)}} = 0.96 \text{ ft.}$$

From Fig. 2 the ratio of the variation of the mean temperature of the slab to the variation of the external temperature is 0.86. Then the departure of the mean temperature of the slab from the mean annual is: (0.86) (22) = 19.0°F.

Similar computations will show the 15 day period to contribute a 2.7°F. change, and the daily period an 0.5°F. change. The maximum departure of the mean temperature of the slab from mean annual temperature would result if these effects should occur simultaneously. Then the maximum departure from the mean would be 19.0°F. + 2.7°F. + 0.5°F. = 22.2°F. as shown on Fig. 5.

TIME LAG OF MEAN TEMPERATURE OF THE CONCRETE BEHIND THE EXTERNAL VARIATION

Maximum or minimum mean temperature is attained by a slab some time after the external temperature has reached the corresponding point.


ESTIMATED VARIATIONS OF MEAN TEMPERATURES OF CONCRETE DAMS EXPOSED TO AIR TEMPERATURES ON BOTH FACES
ESTIMATE BASED ON h² = .050 ft²/hr
Mean Annual Temperature
Departure - Degrees Fahrenheit Above Mean Annual
Below Mean Annual
LEGEND
Northern Station
Southern Station
Thickness of Dam in Feet
FIG. 5

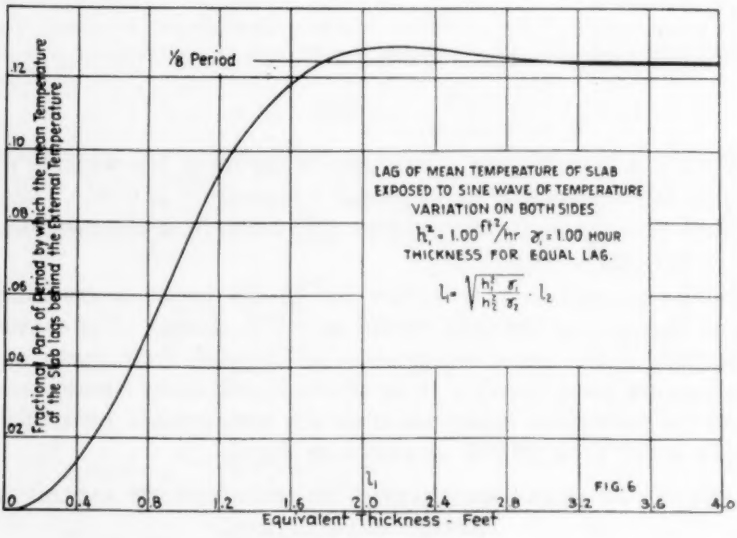

⅛ Period
Fractional Part of Period by which the mean Temperature of the Slab lags behind the External Temperature
LAG OF MEAN TEMPERATURE OF SLAB EXPOSED TO SINE WAVE OF TEMPERATURE VARIATION ON BOTH SIDES
h₁² = 1.00 ft²/hr α = 1.00 HOUR THICKNESS FOR EQUAL LAG.
l₁ = √(h₂²/h₁² · α₁/α₂) · l₂
Equivalent Thickness - Feet
FIG. 6

FIG. 5 AND 6

Fig. 6, shows the fractional part of the period by which the mean temperature of the slab lags behind the corresponding phase of the external temperature. The diagram is computed for slabs of various thicknesses exposed to a sinusoidal temperature variation having a period of one hour. Equivalent lengths are computed from equation (2).

Example 4.

How long will be required for an arch ring 40 ft. thick to reach its minimum mean temperature after the mean daily temperatures have reached their lowest point for the year if $h^2 = 0.050 \frac{ft.^2}{hour}$?

Since the yearly variation has the longest period it will govern the time when the minimum occurs.

Find the equivalent length for $h^2 = 1.00 \frac{ft^2}{hour}$, γ, $= 1.00$ hour from formula (2) thus

$$l_1 = \frac{1.00}{\sqrt{(8760)\ (0.050)}} l_2 = 1.913 \text{ ft.}$$

On entering Fig. 6 with this value the fractional part of the period by which the mean temperature of the slab lags behind the external temperature is found to be 0.127. The lag is then (8760) . (0.127) = 1110. hours.

= 46. days.

Fig. 2 and 6 are calculated for the conditions which exist after the concrete has been exposed to the temperature variations a sufficient time for steady conditions to have become established.

NATURE OF SOLUTIONS

The diagrams accompanying this paper are based upon formulas which are exact for a material which is homogeneous and isotropic, and for which the conductivity and specific heat are constants. The differential equation from which they were derived is of the linear type and guarantees for them the validity of the principal of superposition. It is, therefore, permissible in the case of a dam which is losing setting heat and is at the same time subjected to seasonal temperature variations to picture the temperature changes as consisting of a descent of the mean concrete temperature toward the mean annual temperature which is not interfered with by the external temperature fluctuations and upon which is superimposed the changes due to these fluctuations.

GENERAL SUMMARY

In addition to the explicit information furnished by the diagrams accompanying this paper, the following general information may be obtained:

(1) The time required to lose a given part of the heat of setting is, if other things are equal, proportional to the square of the thickness.

A dam 50 ft. thick will lose approximately 90 per cent of its excess heat in a year and a half. A dam 500 ft. thick will require a century and a half to lose the same proportion.

(2) The seasonal change of mean concrete temperature for dams thicker than about 50 ft. is nearly inversely proportional to the thickness. This fact may be deduced from Fig. 2 and may also be demonstrated by means of the solution for the semi-infinite solid exposed to a variable temperature.* It may be shown by the use of this solution that temperature changes due to varying air temperatures will become imperceptible 50 ft. beneath the surface of the concrete. The rule follows immediately when this fact is known.

(3) After the excess heat is lost all portions of a dam where the thickness is greater than about 35 feet will attain their minimum temperature approximately one and one-half months after the seasonal temperatures reach their lowest point. Since the lowest temperatures for the year occur about January 15, the lowest mean concrete temperatures for the above thicknesses may be expected about March 1. For thinner sections the minimum mean temperature will be attained earlier but in any case every portion of the dam may be expected to reach its lowest mean temperature at some time within the period between January 15 and March 1.

MEASUREMENT OF THE DIFFUSION CONSTANT

The diffusion constant may be found by the following method: A cylinder having a length equal to twice its diameter is cast with a one-half inch diameter hole along the axis and stored in damp sand or under water until ready to be tested. The cylinder is prepared for testing by inserting a long stem mercury thermometer in the hole to a depth which will place the bulb of the thermometer at the geometrical center of the cylinder. The hole in the cylinder should be only deep enough to permit this. Before inserting the thermometer a small wad of asbestos wicking should be pushed to the bottom of the hole to protect the thermometer bulb, and the hole filled with water to provide good thermal contact. The stem of the thermometer above the bulb should be wound with asbestos wicking for mechanical protection and to prevent thermal currents in the hole during testing. The top of the hole should be sealed with plaster of paris. The cylinder is brought to a uniform high temperature by immersing it for several hours in boiling water. It is then transferred quickly to a vat of running cold water and the time history of the temperature at the center observed. The cylinder should be suspended in such a way that the cold water will

*See Carslaw "The Conduction of Heat," page 47. Second edition.

have access to all parts of the surface. A pair of wire loops cast into the top of the cylinder, and to which a wire bail may be attached will serve for suspending the cylinder and will also be of assistance for handling the cylinder in the boiling water. The diffusion constant may be obtained from Fig. 7. An example, based on test data obtained by the writer, will illustrate its use.

A 6-in. diameter cylinder of concrete, 12 in. long, having previously been boiled in water for two hours was quickly transferred to a vat of running cold water. At the time of immersion the temperature of the cylinder was 94°C. throughout.

The following observations are extracted from the test data: At

Time (P. M.)			Temp. at Center of Cylinder	Water Temperature	Remarks
Hr.	Min.	Sec.			
1	50	0.5	94.0°C.	7.0°C.	Time of immersion
1	56	0.0	86.4°C.	5.0°C.	
2	02	0.0	69.0°C.	5.0°C.	
2	08	0.0	49.1°C.	5.0°C.	
2	14	0.0	34.1°C.	5.0°C.	
2	20	0.0	23.8°C.	5.0°C.	
2	26	0.0	16.9°C.	4.7°C.	

2:08:00 P. M. the elapsed time was 0.300 hr. The initial difference was 94.0°C. − 5.0°C. = 89.0°C. The temperature remaining at 2:08:00 P. M. was 49.1°C. − 5.0°C. = 44.1°C. The part remaining at 2:08:00 P. M. was $\frac{44.1}{89.0} = 0.496$. The diffusion constant obtained from Fig. 7 was 0.041 $\frac{ft.^2.}{\text{hour}}$

Fig. 7 may be used with cylinders larger than 6 x 12 in., if geometrically similar, since the law of times as expressed by equation (1) holds for this case also. When used for this purpose l represents the length of the cylinder.

The laboratory staff of the Bureau has determined the thermal properties of concretes made with aggregates representing a wide variety of types. With these data at hand it is possible to estimate the conductivity, specific heat, density and diffusivity if a petrographic analysis of the aggregate and the details of the mix are available. These data and a description of the apparatus and test methods developed for obtaining the thermal properties of concrete are to be published soon in a bulletin to be issued by the Bureau of Reclamation. This bulletin and the one previously mentioned will appear as com-

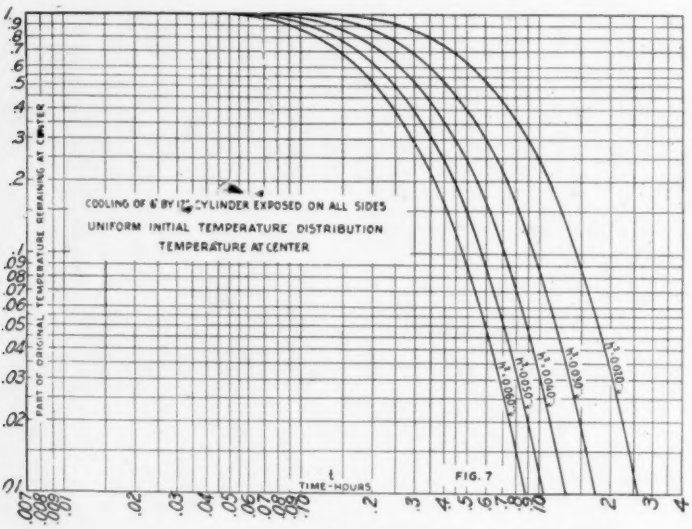

COOLING OF 6" BY 12" CYLINDER EXPOSED ON ALL SIDES
UNIFORM INITIAL TEMPERATURE DISTRIBUTION
TEMPERATURE AT CENTER
PART OF ORIGINAL TEMPERATURE REMAINING AT CENTER
TIME-HOURS
FIG. 7

FIG. 7

panion volumes to present the basic laboratory data and the formulas necessary to apply the data to actual problems.

The problems treated in this paper require only that the diffusivity be known, however, a more nearly complete knowledge of the thermal properties is essential in many other cases.

ACKNOWLEDGMENTS

The data contained in this paper are extracted from office memoranda of the United States Bureau of Reclamation, which were prepared by the writer, under the direction of Byrum W. Steele, Senior Engineer, in connection with the design of the Boulder dam. All designing work and design studies are made under the direction of J. L. Savage, Chief Designing Engineer. All engineering and construction work of the Bureau of Reclamation is under the direction of R. F. Walter, Chief Engineer, with headquarters at Denver, Col., and all activities of the bureau are under the general charge of Dr. Elwood Mead, Commissioner, with headquarters at Washington, D. C.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for May-June 1935. Discussion should be available to the Secretary by April 1, 1935.*

Strength and Volume Change of Steam-Cured Portland Cement Mortar and Concrete*

BY CARL A. MENZEL†

INTRODUCTION

In the last 25 years several investigators have reported results of studies of concrete specimens cured within a pressure-tight cylinder in an atmosphere of saturated steam.[1] These studies indicated, in general, that under favorable conditions high pressure steam curing offered possibilities for producing concrete having higher early strength, and increased sulfate resistance, in comparison with concrete cured at normal temperatures. Some of the studies indicated also that lower volume change with changes in moisture content were attained with steam-cured concrete. Some of the investigators found for the steam curing, strengths somewhat higher than for all moist curing. Others found no improvement. The type of aggregate used, particularly as to whether it is siliceous or non-siliceous in nature has been shown to be an important factor in the strength results obtained. In view of the results available, it appeared desirable to make further studies to bring out sufficiently for practical application to the manufacture of cement products the possibilities and limitations of this method of curing. The present investigation was undertaken with this objective in the Research Laboratory of the Portland Cement Association, Chicago. It is concerned primarily with the strength and volume changes of cement pastes and mortars, and mixtures of cement and finely divided silica in both neat and mortar specimens, cured in saturated steam at high temperatures for different periods. Companion moist-cured specimens were made and subjected to the same tests as those which were steam cured.

Difference between Steam-Cured and Moist-Cured Concrete. Concrete cured in steam at high temperatures differs from moist-cured concrete in the extent to which the primary chemical reactions between the cement and the water are supplemented by secondary reactions between the calcium hydroxide liberated during the hydrolysis and

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[1]See accompanying selected list of references.

hydration of the cement and the siliceous particles of aggregate in the mix. That some reaction can take place even in moist-cured concrete between certain siliceous materials and the calcium hydroxide in the hardening paste is recognized, but it requires long periods of moist curing and the strengths developed are lower than those obtained with curing in high pressure steam. The importance of these secondary reactions depends upon the amount of finely divided silica present, its gradation, and the temperature and period of steaming. The test results show that these may be the principal factors determining the properties of steam-cured concrete.

DISCUSSION OF RESULTS—STRENGTH

For a detailed study of the principal results thus far obtained from these studies, the reader is referred to the several diagrams and tables accompanying the text. Details of the test procedure are given in the Appendix. The text is limited to a discussion of the general features of the results. It should be pointed out here that the steam was always saturated and that the specimens were usually cured in the molds 24 hours before placing in the steaming chamber.

Because of the important effect of finely divided silica on the properties of steam-cured concrete, the other factors, such as quantity of mixing water, type and grading of coarse aggregate, composition of the cement, and temperature and duration of the steam curing period, cannot be treated separately. They must be considered in the light of the presence or absence of such silica. Hence, the discussion hinges largely upon this major factor.

Influence of Temperature and Duration of Steaming Period. The effect of variations in temperature and duration of steaming period can be seen in several of the diagrams. The differences that result from varying either of these factors are brought out completely in Fig. 1 and 2. In Fig. 1 the three diagrams at the left show results of steaming with various percentages of finely divided silica (Ottawa sand ground to pass the No. 200 sieve), and those at the right, show comparable curves for an inert material—feldspar. The heavy dash lines in each diagram represent the 28-day strength of moist-cured specimens which are otherwise identical. The cement used was a laboratory mixture of equal parts by weight of 4 brands of normal portland cement.

The significant feature of the results shown in Fig. 1 was the high strength obtained with mixtures containing 30 to 50 per cent of silica when steamed at 350°F. For steaming periods of 16 hr. to 3 days these ranged from 20 to 50 per cent greater than the strengths of the neat cement alone when moist cured for 28 days, and from 2 to 3 times the

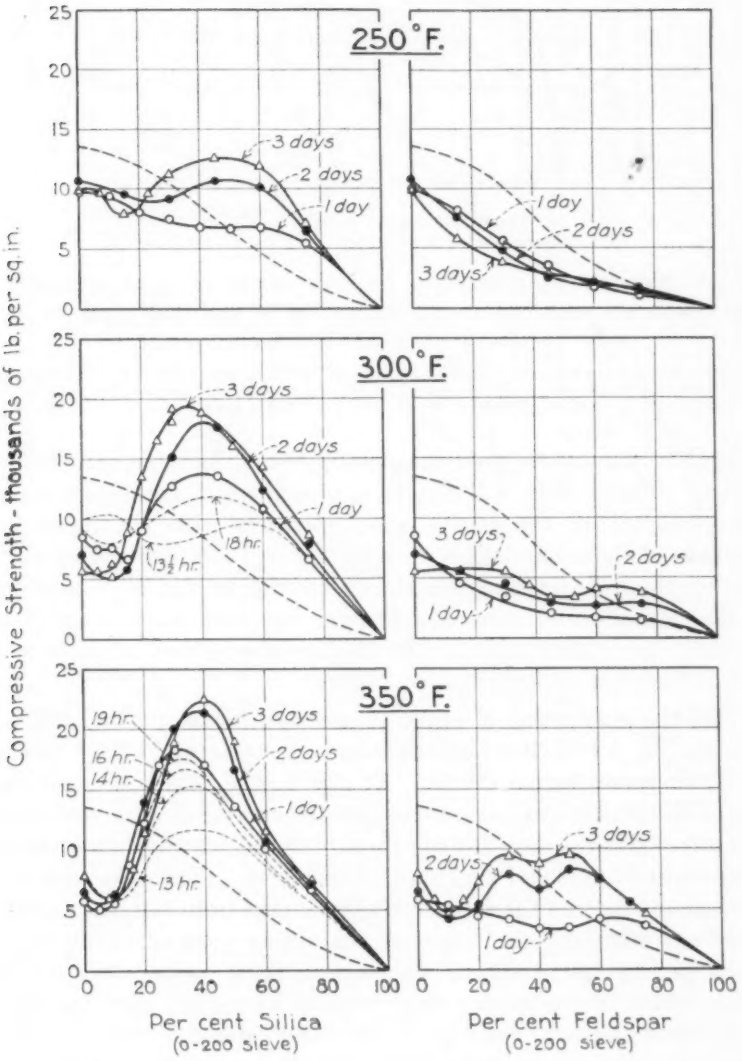

FIG. 1—STRENGTH OF STEAM-CURED CEMENT-SILICA AND CEMENT-FELDSPAR MIXTURES AFTER DIFFERENT STEAMING TREATMENTS

Abscissas represent per cent (by weight) of silica or feldspar in the mixtures. Heavy dash lines represent 28-day strengths of cubes cured in moist room at 70°F. All steam-cured cubes in molds 24 hr. before steaming. See Table 1, appendix, for length of time at maximum temperature. Cement: Laboratory mixture of 4 brands.

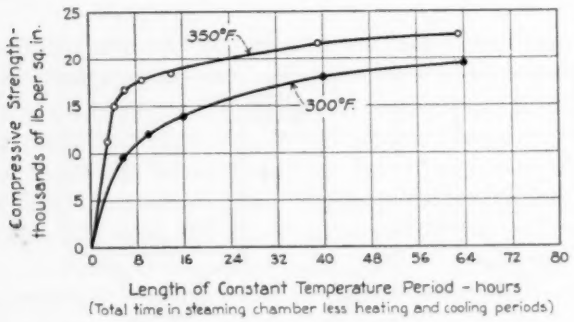

FIG. 2—EFFECT OF DURATION OF EXPOSURE TO HIGH STEAM TEMPERATURES ON THE STRENGTH AT OPTIMUM PERCENTAGES OF SILICA IN THE MIXTURES

Strengths taken from curves in diagrams for 300 and 350°F. in Fig. 1 for optimum percentages of silica (Ottawa sand 0-No. 200). The lengths of the heating, cooling, and constant temperature periods at 300 and 350°F. for a given nominal steaming cycle are given in appendix.

strength of the steam-cured neat specimens. The important influence of finely divided silica will also be apparent from a comparison of the high strengths of the cement-silica mixtures with the much lower strengths of the cement-feldspar mixtures at the various temperatures and durations of steaming periods represented in Fig. 1. The strength of the steam-cured cement-feldspar mixtures was usually substantially lower than that of the 28-day moist-cured strength except when steamed for 2 or 3 days at 350°F.

From the standpoint of optimum strength development with silica present, Fig. 1 indicates the advantage of steaming at 350° rather at 300°F. or lower temperatures. At 350°F. the optimum strength of cement-silica mixtures (with 0-No. 200 Ottawa sand) increased sharply with increase in the duration of exposure to this maximum temperature up to about 8 hours and very slowly thereafter. This is shown in Fig. 2 on which are plotted the compressive strengths from Fig. 1 for different periods of steaming for mixtures containing approximately the optimum percentage of silica. In plotting these values only the time during which the specimens were exposed to the full 350° (or 300°F.) temperature is used. In Fig. 1 and other figures of this report, the steaming period shown is the total time in the steaming chamber. It will be seen from Fig. 2 that at 350°F. the development of strength was so rapid that with a full 8 hours exposure at this temperature a strength was attained equal to 85 per cent of that developed by exposure for a full 24 hr. At 300°F. the strength with 8 hr. exposure was about 65 per cent of that at 350°, and it required about 40 hr. at 300° to equal the 8-hr. strength at 350°F.

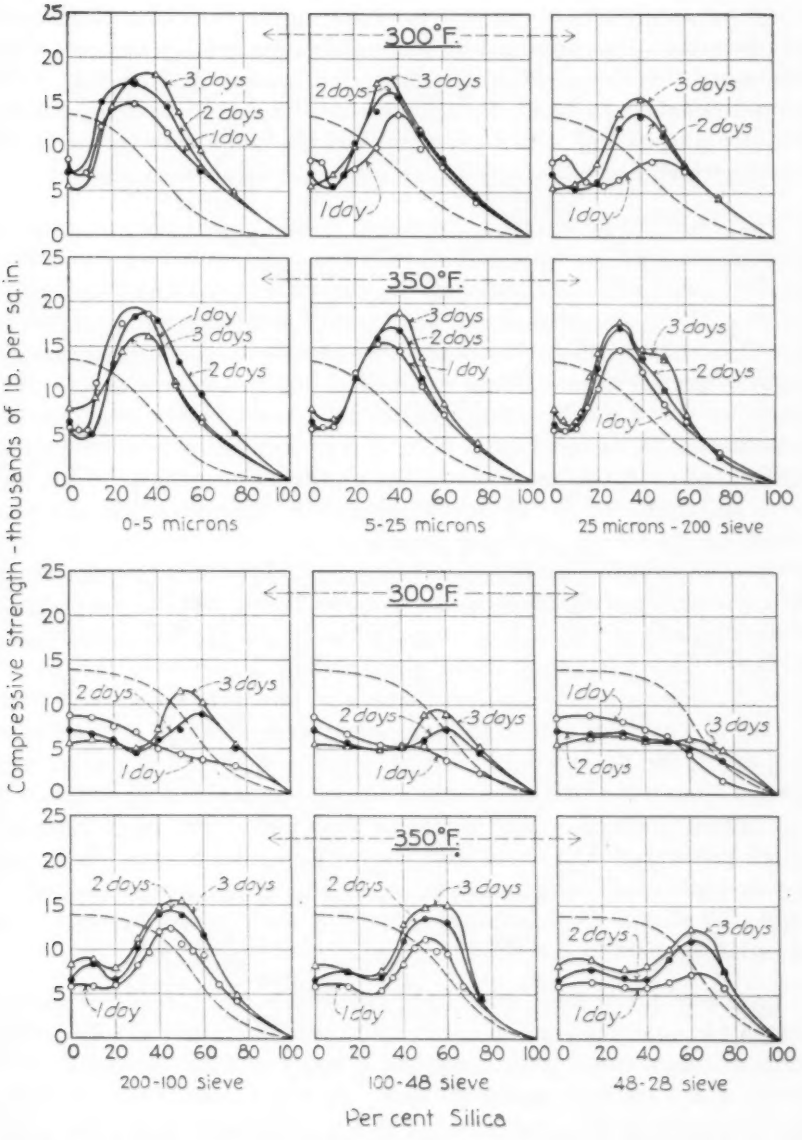

FIG. 3—STRENGTH OF STEAM-CURED CEMENT-SILICA MIXTURES MADE WITH SILICA OF 6 SIZE RANGES BETWEEN 0 AND NO. 28 SIEVE

Dash line curves represent 28-day strengths of cubes cured in moist room at 70°F. All steam-cured cubes in molds 24 hr. before steaming. See Table 1, appendix, for length of time at maximum temperature. Cement: Laboratory mixture of 4 brands.

These data are of interest in indicating a desirable steaming cycle for the manufacture of concrete products on the basis of a 24-hr. cycle of operation. By allowing 4 hr. for handling material in and out of the steaming chamber, and 10 to 12 hr. for heating and cooling, there would remain 8 to 10 hr. for exposure at the maximum temperature. By steaming at 350°, this 8 to 10 hr. exposure should give the optimum results which can be expected with a given cement-silica mixture.

Influence of Size of Silica Particles. Fig. 3 shows the compressive strengths of various cement-silica mixtures steam-cured for 1 to 3 days at 300° and 350°F. for 6 different size ranges between 0 and the No. 28 sieve. These diagrams show that for any given steaming treatment the highest strengths were obtained with silica in the three size ranges from 0-No.200 sieve. When steamed at 300°F. the presence of silica particles above the No. 200 sieve did not improve the strength of the mixture unless steamed for periods of 2 to 3 days. When steamed at 350°F. however, the presence of silica particles larger than the No. 200 sieve and up to the No. 48 sieve improved the strength of the mixtures substantially for all steaming periods shown. Particles coarser than the No. 48 sieve did not improve the strength of the mixtures materially unless steamed at 350°F. for periods of 2 or 3 days. These comparisons show that size of the silica particles is one of the factors contributing to the increase in strength under steam curing. That other factors are involved is indicated by the fact that higher strengths are shown by the curves of Fig. 1 for graded mixtures than were obtained with silica in the size range 0-5 microns. In the mixture in Fig. 1 the silica was graded from 0-No. 200 sieve, and contained only about 15 per cent of 0-5 micron material. The mixtures with 0-No. 200 silica required much less water than those with the very fine 0-5 micron silica, and resulted in a much denser paste structure which accounts for the higher strengths obtained. A complete study of these other factors is left for a later report.

Tests with Silica Bearing and Inert Materials. The excellent results shown for mixtures containing 30 to 50 per cent finely divided silica (Ottawa sand, 0-No. 200) suggested a study of other aggregate materials which are only partly siliceous. In Fig. 4, results of these tests are shown for 28 days moist curing at 70°, and steam-curing for 1 day at 300° and 350°F. Included with such silica bearing materials as granite, Haydite (burnt shale), lava, cinders, blast-furnace slag, common glass, and flue ash, were the inert materials: limestone, marble, and feldspar. All the materials were ground to pass the No. 200 sieve and were treated in the same manner as the 0-No. 200 Ottawa

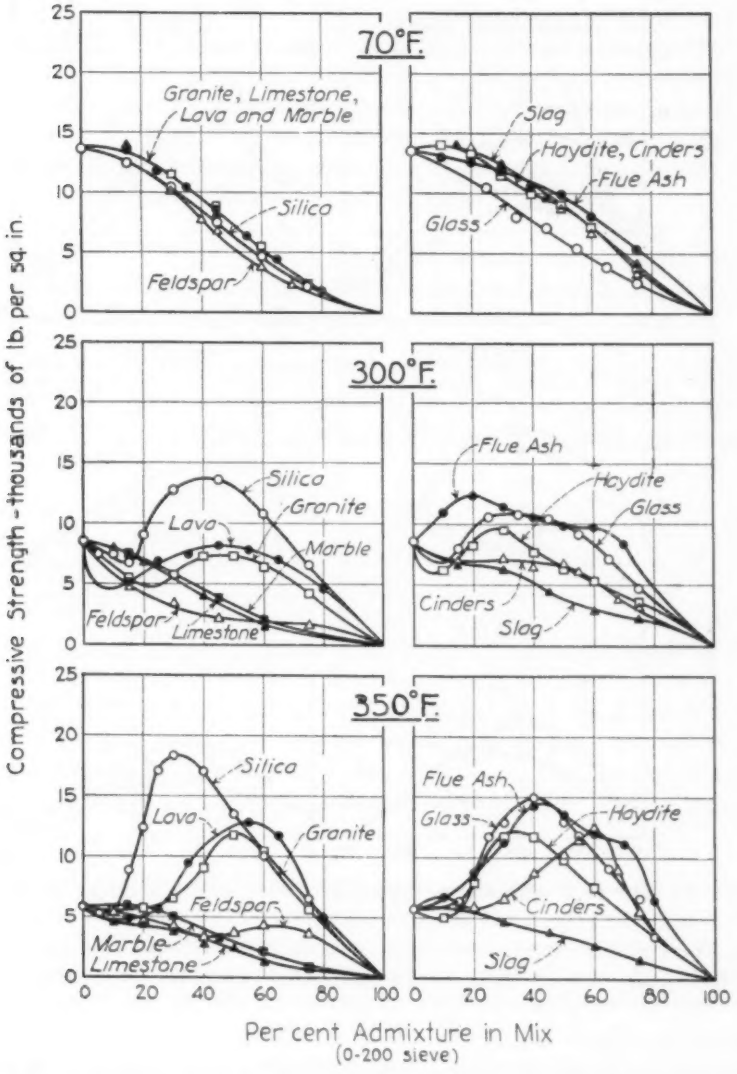

FIG. 4—STRENGTH OF MOIST-CURED AND STEAM-CURED MIXTURES OF CEMENT AND VARIOUS FINELY-DIVIDED MATERIALS

Curves for 70°F. represent strength of mixtures moist-cured for 28-days; those for 300 and 350°F. represent the strength of mixtures in steaming chamber 1 day after 24 hr. in the molds. Compositions of admixture materials are given in Table 2, appendix. Cement: Laboratory mixture of 4 brands.

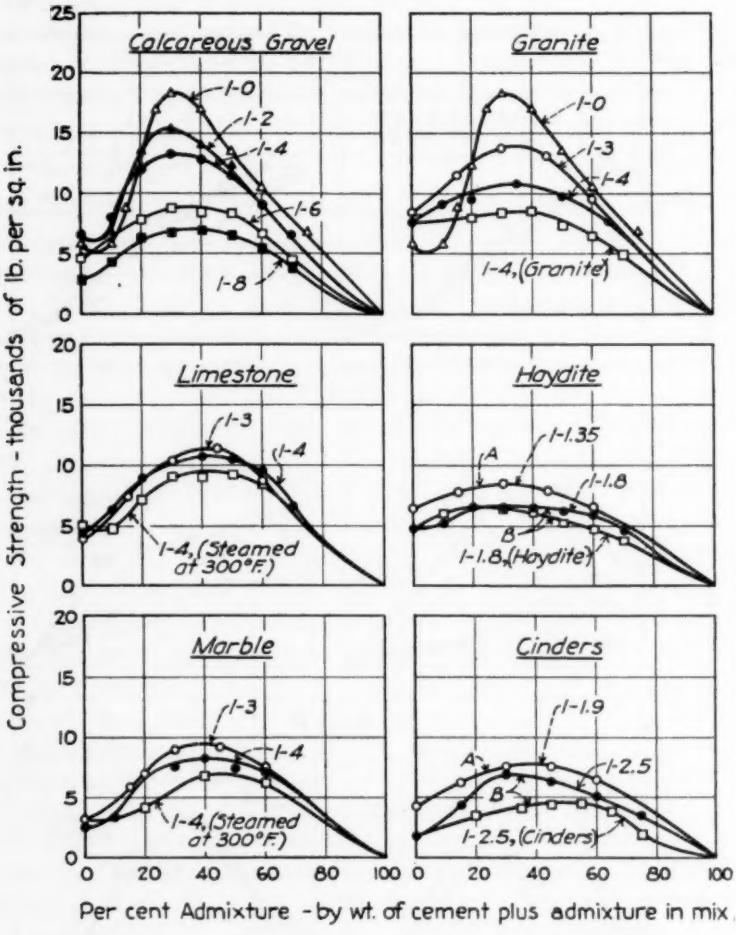

FIG. 5—EFFECT OF TYPE AND QUANTITY OF 0-NO.200 SILICEOUS MATERIAL IN THE CEMENT PASTE ON THE STRENGTH OF CONCRETE OF VARYING AGGREGATE CONTENT STEAMED AT 350°F. FOR 1 DAY AFTER 24 HR. IN MOLDS

The ratios in the diagram indicate the proportions of cementing material to aggregate. In all cases the cementing material includes the percentage of admixture indicated by the abscissa. Unless otherwise indicated the admixture is the ground Ottawa silica. All admixtures were graded 0-No. 200 sieve. In the diagrams for Haydite and Cinders the curves labeled A and B represent mixes which are comparable on a volume basis, respectively, to those labeled "1:3" and "1:4" in the other diagrams for the natural aggregates. The gravel was graded as follows: 15% by weight, No. 48-28; 10% No. 28-14; 15%, No. 14-8; 20%, No. 8-4; and 40%, No. 4-¾ in. The other aggregates were graded: 20%, No. 48-28; 15%, No. 28-14; 15%, No. 14-8; 20%, No. 8-4; and 30%, No. 4-⅜ in. Cement: Laboratory mixture of 4 brands.

sand in the tests shown in Fig. 1 and 3. For comparison, the curves for this pure silica are shown in Fig. 4 also.

It will be seen from Fig. 4 that the optimum percentages of the various active materials and the strengths obtained varied somewhat. In general, the optimum percentages were higher and the strengths lower than for the ground Ottawa sand. Such data as are available indicate also that the various silica bearing materials are influenced by particle size, temperature, duration of steaming period, etc., in much the same manner as the ground Ottawa sand.

It is of interest to note that for moist curing, the artificial silica bearing materials generally show slightly higher strengths for a given mixture than the natural silica bearing materials.

The slag used in these tests was the only siliceous material which did not react under the steaming treatment as did the other silica bearing materials. In these tests it produced results comparable with those from the limestone and marble.

Combinations with No. 48-⅜-in. Aggregates. In Fig. 5 are shown the results of tests in which various aggregates are combined with cement-silica mixtures of varying silica content. The proportions of cement-silica material to aggregate varied from 1:0 to 1:8 by weight. The proportions of silica (ground Ottawa sand 0-No. 200) in the cement-silica mixture varied from 0 to 75 per cent through about the same percentage intervals as in the tests shown in Fig. 1 to 4 where the cement-silica mixtures were used without aggregates. Six different aggregates were used, all graded No. 48-⅜-in. The minimum size was fixed at the No. 48 sieve to eliminate all particles in the active size range.

In the case of three of the aggregates: granite, Haydite, and cinders, fine material (0-No. 200) of the same type as the aggregate itself was used in place of the ground Ottawa sand in one of the mixes. These three mixes are indicated on the diagram by their proper designation.

The curves for the 1-0 mixtures in the two upper diagrams (taken from Fig. 4, for the 350° steaming) show the strengths obtained from cement-silica mixtures without aggregate. It will be seen that the other curves showing strengths with the various aggregate mixtures are, in general, similar in characteristics indicating the close relationship between the strength of the active binding medium and the concrete made with different proportions of aggregate.

For corresponding mixes (1:3 or 1:4) the strengths for the different aggregates at the same percentages of active silica were not identical. The limestone and marble showed somewhat lower strength than the calcareous gravel or granite and the Haydite and cinders still lower. The curves marked A and B (Fig. 5) from these latter two aggregates correspond on a volume basis with the 1:3 and 1:4 mixes, respectively, for the other aggregates. The lower strength in the case of limestone and marble is believed to be due: (1) to a lower bond between the cementing paste and aggregate particles consisting wholly of calcareous material; and (2) to a slight softening of the marble when exposed to steam at high temperatures. With the exception of marble, it is believed that the effect of high pressure steam on the strength of the different aggregate materials is of no practical significance.

The relatively low strength shown by the Haydite and cinders is a reflection of the lower strength of the aggregate particles. It must not be overlooked that the strengths obtained in these various tests are very high in terms of ordinary concrete experience. The very lowest value on Fig. 5 is 2000 p. s. i.—a strength still commonly used by engineers as a basis for structural design. This particular value is for a mixture 1:2.5 by weight in which the cementing material is a 25:75 mixture of cement and ground cinders, and the coarse aggregate is cinders graded No. 48-3/8 in.

Effect of Silica in the Aggregate Fines. In Fig. 6 are presented some tests to show the effect of the silica content in the finer sizes of aggregate. The same aggregate types as were used in the tests of Fig. 5 are included. In this case, however, the cementing material consisted entirely of portland cement and the only active silica available for combination, was in the 0-No. 28 portion of the aggregates.

Of the six aggregates shown in Fig. 6 those in the three diagrams at the left—calcareous gravel, limestone, and marble—are more or less inert, while those in the three diagrams at the right—granite, Haydite, and cinders—are silica bearing.

Each curve in the diagram is identified by the percentage and character of material composing the 0-No. 28 portion of the aggregate. The curves themselves represent the variation in strength for variations in quantity of aggregate, the mixes varying from neat cement to 1:10 by weight. Significant comparisons are offered in these diagrams between corresponding points on the different curves, as these show the effect on the strength of a particular mix of a change in amount or character of the 0-No. 28 portion of the aggregate.

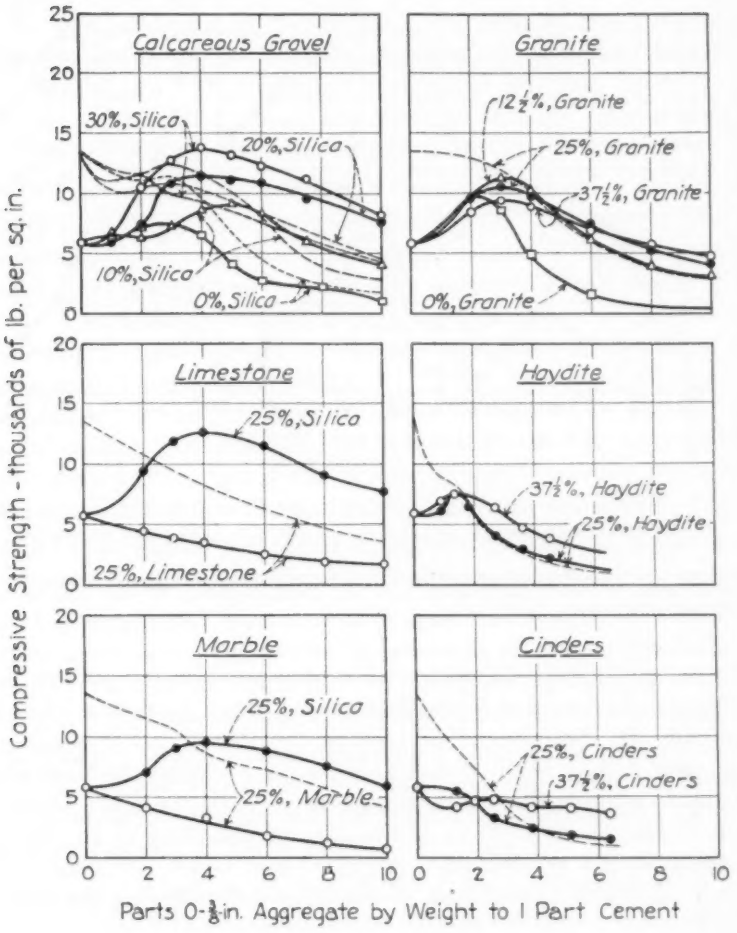

FIG. 6—EFFECT OF TYPE AND QUANTITY OF 0-NO. 28 MATERIAL IN THE AGGREGATE MIXTURE ON THE STRENGTH OF CONCRETE OF VARYING CEMENT CONTENT STEAMED AT 350°F. FOR 1 DAY AFTER 24 HR. IN MOLDS

Dash line curves represent 28-day strengths of cubes cured in moist room at 70°F. Percentage values in each diagram refer to weight of graded 0-No. 28 material, of the type indicated, in the aggregate mixture. For example, in the lower left diagram the curve labeled "25%, silica" means that the 0-⅜ in. aggregate mixture consisted of 25% by weight of silica, 0-No. 28 in size, and 75% of marble, No. 28-⅜ in. Similarly, the curve labeled "25%, marble" means that the aggregate mixture consisted of 25% by weight of marble 0-No. 28, and 75% of marble, No. 28-⅜ in. Data on the weight and grading of aggregates are given in Table 3, appendix. Cement: Laboratory mixture of 4 brands.

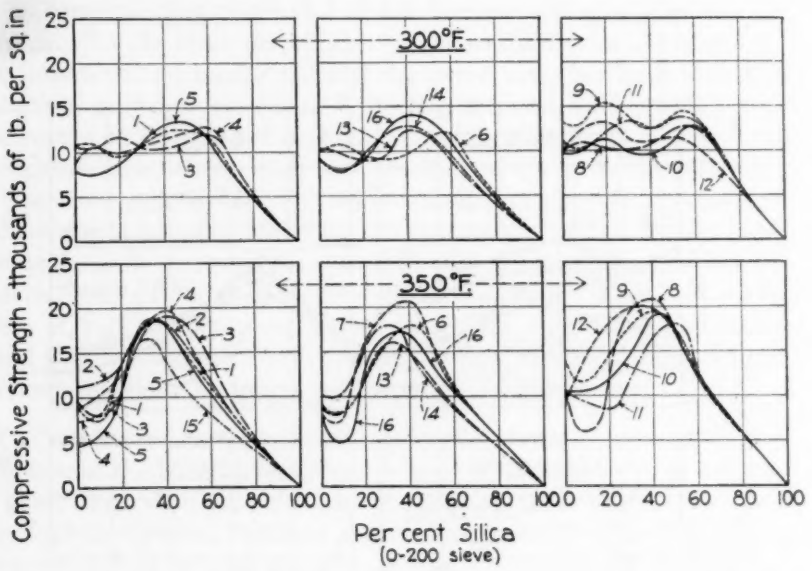

FIG. 7—STRENGTH OF STEAM-CURED CEMENT-SILICA MIXTURES MADE WITH 16 COMMERCIAL CEMENTS AND 0-NO. 200 OTTAWA SAND

All curves based on strength of cubes steamed for 1 day at 300 or 350°F. after 24 hr. in molds. The two left diagrams for each temperature are for normal portland cements; the right diagram for high early strength cements.

These results show the importance of controlling the grading, composition, and amount of 0-No. 28 material in the aggregate when concrete is to be steam-cured at high temperatures and pressures. In some cases it may even be desirable for the concrete products manufacturer to remove all silica or silica bearing particles in the active size range (0-No. 48 sieve) from the aggregate and then to combine the aggregate with a cement-silica mixture (with 0-No. 200 silica) of the desired quality in rich or lean proportions depending on the strength desired.

Characteristics of the Cement. On Fig. 7 are shown results of tests with 16 different cements—11 normal portlands and 5 high-early strength cements. These tests, which were for 1-day steaming at 300 and 350°F. are similar to those shown in Fig. 1 for the 1-day steaming at corresponding temperatures. In both cases the silica is the ground Ottawa sand, 0-No. 200. It will be noted that the curves for the different cements in Fig. 7 are, in general, similar among themselves and similar to the curves in Fig. 1 for which a laboratory mixture of four brands of normal portland cement was used.

For the two lower diagrams at the left, which represent results of normal portland cements, maximum strengths are shown for percentages of silica between about 30 and 50 just as in previous figures. For the lower diagram at the right, which includes the 5 high-early strength cements, the increases in strength with percentage of silica are more irregular. The maximum strengths attained, however, are of the same order as for the normal portland cements. The curves for 300° steaming indicate that steaming for 1 day at this temperature was insufficient to develop the full potential strength of the mixtures.

Using 12 of the 16 cements represented in Fig. 7, the various cement-silica mixtures were combined with aggregate in a series of tests to determine the strength of concrete after curing in steam at 350°F. for 8 hr.* The concrete consisted of 1 part of cement-silica mixture (proportion of cement to silica variable) to 4 parts by weight of calcareous gravel graded No. 48-⅜ in. These concrete mixtures corresponded to the 1:4 mixtures for calcareous gravel in Fig. 5. One group of specimens was steamed in the molds within 2 hr. after molding while a second group was steamed upon removal from the molds after 24 hr. of moist curing. These tests showed:

(1) A close relationship between the strengths of the concrete and the cement-silica mixtures alone for a given cement,

(2) The same rapid development of concrete strength with optimum percentages of silica during the 8-hr. steaming period, as was found for laboratory cement (a mixture of 4 brands) in combination with the silica, (See "Influence of Temperature and Duration of Steaming"),

(3) Strengths at optimum percentages of silica of from 10,000 to 13,000 p. s. i. for the different cements for the specimens steamed immediately after molding and of from 11,500 to 14,000 p. s. i. for specimens steamed at age of 24 hr.

It appears, therefore, that with an optimum percentage of silica (ground 0-200), high strengths can be obtained with any of the different brands of commercial portland cements by curing in steam at 350°F. for a full 8 hours; also that steaming can begin several hours after molding or at the age of 24 hr. as desired.

Amount of Water Used. Studies in which the quantity of water was varied showed that for any given cement-silica mixture an increase in the water in the active cementing paste decreased the strength of the steam-cured product in the same manner as for ordinary concrete cured in normal temperatures.

*The 12 cements used comprised 8 normal portland and 4 high early strength cements. Results are not available for 4 of the 16 cements, (Cements 1, 2, 11 and 15), because of lack of material.

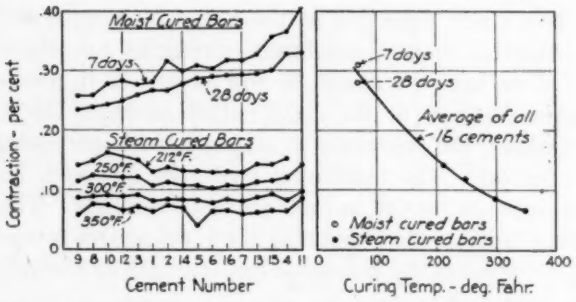

FIG. 8—EFFECT OF CURING OR STEAMING TEMPERATURE ON VOLUME CHANGE OF NEAT BARS MADE WITH 16 COMMERCIAL CEMENTS

Cement in left-hand diagram arbitrarily arranged in order of increasing contraction for bars moist-cured 28 days. Steaming periods in all cases were for 1 day after 24 hr. in molds.

DISCUSSION OF RESULTS—VOLUME CHANGE

The results of the volume change tests are given in Fig. 8 to 11. In all cases the contraction referred to is that between the saturated and air-dried condition following either a period of moist curing or steaming at 1 day after 24 hours in the molds. Steam-cured specimens were all saturated by submersion in water for 28 days after the steam-curing. Measurements on the moist-cured specimens began immediately upon removal from the moist room. The air drying was carried out at 70°F. and 35 per cent relative humidity.

Volume Change of Mixtures Without Silica. Steam-cured neat cement bars made with 16 different cements all showed a progressive decrease in contraction with a progressive increase in the curing temperature from 212 to 350°F. (See Fig. 8). When cured for the nominal period of 1 day at 350°F., the contraction of the neat bars varied from ⅛ to ¼ that of the moist-cured bars depending on the cement used; the average for all cements was about 1/5. In general, the contraction of these bars was closely proportional to the strength of corresponding neat cement cubes steamed at 350°F. for the same period. Thus, it appears that with curing at 350°F. those factors which increase strength of a neat cement paste also increase volume change. In the absence of silica the volume change of steam-cured concrete made with a given cement depends mainly on the volume of the paste, since the volume change of most aggregates is usually negligible and was not found in these tests to be influenced to an important degree by the steaming treatment.

Comparison between concrete and neat cement specimens from the laboratory mixture of 4 brands, shows that in the complete absence of silica, moist-cured concrete contracted about 7 times as much as the

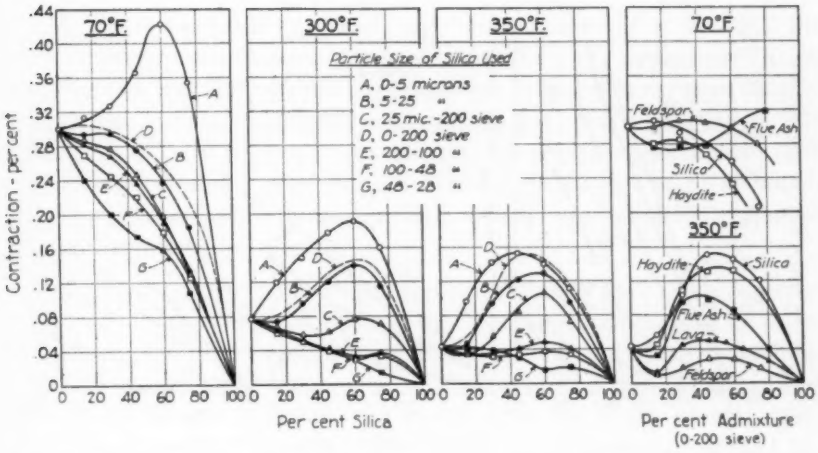

FIG. 9—EFFECT OF FINENESS AND TYPE OF SILICA ON VOLUME CHANGE

Contractions for 70°F. are for bars moist-cured 7 days; contractions for 300 and 350°F. are for bars steamed 1 day after 24 hr. in molds. Cement: Laboratory mixture of 4 brands.

concrete steamed at 350°F. On the basis of the neat cement tests shown in Fig. 8, it seems probable that the volume change of concrete steamed at 350°F. will range from about ⅛ to ¼ of that of moist-cured concrete, depending on the cement used.

Volume Change of Mixtures with Silica and Silica Bearing Materials. In general, the curves representing the contraction of different cement-silica mixtures when steam-cured, parallel rather closely the curves representing the strength characteristics (compare Fig. 9 and 4). The addition of more than 10 per cent of silica in the active size range to the cement paste resulted in a rather sharp increase in contraction which was greatest with silica contents between 40 to 60 per cent depending on the particle size used. Further additions of silica usually resulted in much lower contractions.

From the foregoing it appears that over a considerable range of silica additions an increase in strength is accompanied by an increase in volume change of the steam-cured mixture. It is important to point out, however, that the volume change curves for the various cement-silica mixtures steamed at 350° were generally similar to those steamed at 300°, although the strengths of the mixtures steamed at 350°F. were substantially higher. This indicates that the full advantage of high-early strength may be realized by steaming at 350° without developing volume changes higher than those of similar cement-silica mixtures steamed at 300°F.

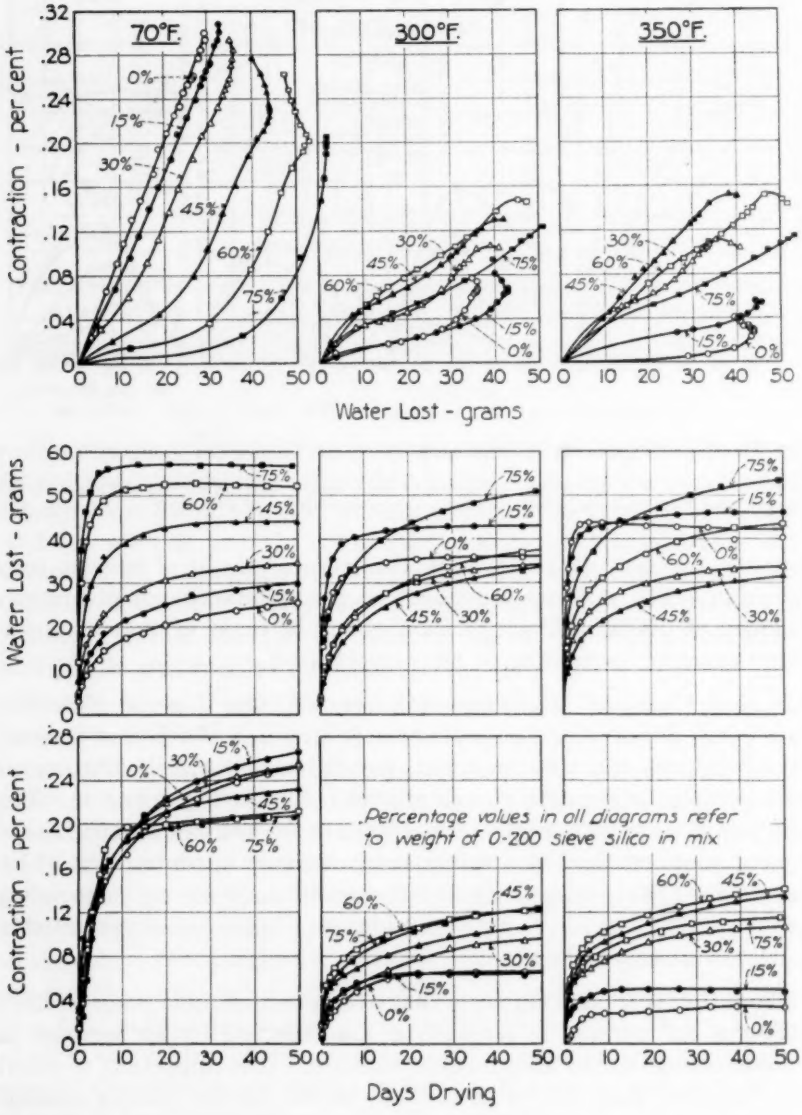

FIG. 10—CONTRACTION AND WATER-LOSS OF MOIST-CURED AND STEAM-CURED BARS OF NEAT CEMENT-SILICA MIXTURES

Curves for 70°F. represent properties of mixtures moist-cured for 7 days; those for 300 and 350°F. represent properties of mixtures steamed for 1 day after 24 hr. in molds. Cement: Laboratory mixture of 4 brands.

The data available indicate that when steamed at 350°F., the volume change of mixtures of cement and finely divided silica bearing materials (0-No. 200) such as Haydite, flue ash, and lava, were usually lower than for similar mixtures of cement and fine silica (0-No. 200 Ottawa sand). With the lava mixtures the greatest contraction (See Fig. 9) exceeded only slightly that of the neat cement alone although the strengths at optimum percentages of lava were over twice that of the neat cement (See Fig. 4).

It appears from all of the volume change studies that when concrete containing finely divided siliceous materials is steamed at 350°F. its volume change will range from about ⅛ to ½ of that of corresponding mixtures of moist-cured concrete depending on the type, grading, and amount of siliceous material, and on the character of the cement used. With the knowledge that at 350°F., particles of siliceous material below the No. 48 sieve are active, and above the No. 48 sieve are relatively inert, mortars can be designed with 0-No. 28 material to have high strength with low volume change. In this manner it has been possible to reduce the differential shrinkage between the coarse aggregate and the mortar sufficiently so that no surface checks were formed even though the concrete was dried rapidly from a saturated to a bone-dry condition in an oven at a temperature of 130-140°F. Thus it appears that steam-cured concrete of high strength and low volume change can be produced which will not craze or surface check under severe service conditions.

GENERAL COMMENTS ON SOME OTHER FEATURES OF THE STUDY

Influence of Age of Concrete When Steamed. In general, the specimens were steamed after curing in the mold for 24 hr. at 70°F. This preliminary curing was necessary to harden the plastic specimens sufficiently so that the leanest mixes could be removed from the mold and placed in the steaming chamber without damage. In a few instances, however, the concrete strength specimens were steamed in the mold within 1 or 2 hr. after molding. In other instances the steaming was delayed until the specimens had moist cured for 3, 5, 7 and 28 days. The results indicated that the strength of steam-cured concrete, with or without silica, is not influenced to any important degree by the age of the concrete when exposed to the steaming treatment at 350°F. (Data for lower temperatures are not available.) Such data as are available indicate that similar remarks may be made with regard to the volume change properties of concrete steamed at ages ranging from 1 day to over two months. Data for ages less than one day are not available.

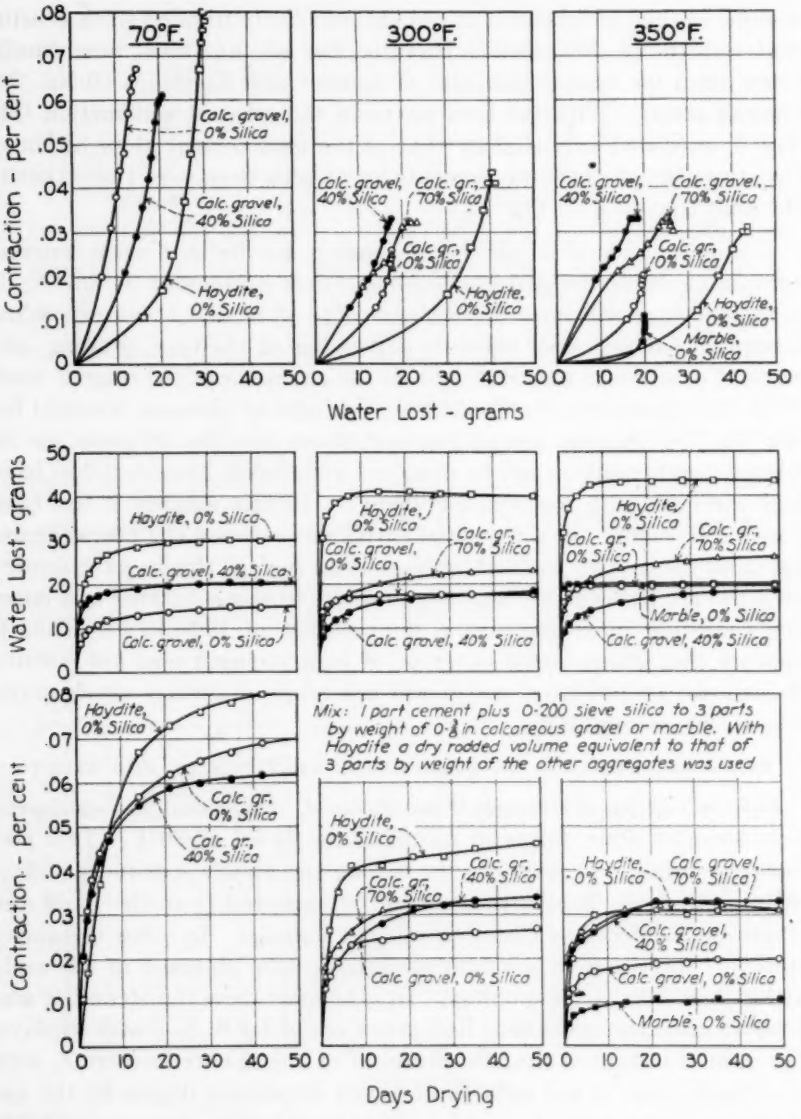
70°F.
300°F.
350°F.
Contraction - per cent
Water Lost - grams
Days Drying
Calc. gravel, 0% Silica
Calc. gravel, 40% Silica
Haydite, 0% Silica
Calc. gr., 70% Silica
Calc. gr., 0% Silica
Calc. gr., 40% Silica
Marble, 0% Silica
Mix: 1 part cement plus 0-200 sieve silica to 3 parts by weight of 0-¾ in calcareous gravel or marble. With Haydite a dry rodded volume equivalent to that of 3 parts by weight of the other aggregates was used

FIG. 11—CONTRACTION AND WATER-LOSS OF MOIST-CURED AND STEAM-CURED BARS OF CONCRETE WITH CEMENT OR CEMENT-SILICA MIXTURES

Curves for 70°F. represent properties of concrete moist-cured for 7 days; those for 300 and 350°F. represent properties of concrete steamed for 1 day after 24 hr. in molds. Cement: Laboratory mixture of 4 brands used without admixtures or with amount of silica (ground Ottawa sand 0-No. 200) indicated. Aggregates graded 0-No. 28, 11%; No. 28-14, 11%; No. 14-8, 11%; No. 8-4, 11%; No. 4-¾ in., 56% (by weight).

Although it appears that concrete may be successfully steamed at early ages there is some question about the practicability of steaming plastic concrete in the mold. In the laboratory studies of steaming at early periods a considerable bond developed between the concrete and the metal mold which made stripping of the mold difficult and damaged the surface of the concrete. The appearance of the surface of the concrete was marred also by grease used to lubricate the interior surfaces of the mold. On the other hand, there appears to be no good reason why mixes from which the molds can be removed immediately after compacting, such as power-tamped or vibrated concrete, cannot be steamed successfully as early as desired.

Leaching and Efflorescence. As mentioned previously, one of the products resulting from the hydration of cement at ordinary temperatures is calcium hydroxide. This is somewhat soluble in water and in porous concretes is frequently precipitated in the form of calcium carbonate on the surface of drying concrete as a white deposit, causing unsightly blemishes termed "efflorescence." Calcium hydroxide is also liberated during the hydration of steam-cured cement paste, but in apparently larger amounts. The leaching of calcium hydroxide is quite apparent even after short periods of immersion in water. It was found, however, that the leaching was very substantially reduced when the cement paste contained finely divided silica (0-No. 200) in the same or nearly the same amounts as required for optimum strength. It is believed that when cured in high pressure steam the silica combines with most of the soluble calcium hydroxide to form a fairly insoluble compound which contributes to the permanent strength and denseness of the hardened cementing paste. Thus, both the amount of soluble material available and its rate of leaching are substantially reduced.

Another effective method for dealing with calcium hydroxide was found by steaming the concrete in the presence of carbon dioxide gas. In this case the carbon dioxide combined directly with the calcium hydroxide to form the relatively insoluble calcium carbonate. With effective carbonation the blemish or loss of color value of brightly colored mortar by efflorescence was completely eliminated in the few cases tried.

Rate of Moisture Loss and Contraction. In general, the rate of moisture loss and the rate of contraction was markedly different for steam-cured material than for similar moist-cured material. For example, the rate of moisture loss of steam-cured neat cement was considerably higher and the rate of contraction considerably lower than that of moist-cured neat cement. The addition of silica to the

steam-cured cement paste reduced the rate of moisture loss substantially, but increased the rate of contraction. This is exactly the reverse of the moist-cured specimens where the addition of silica increased the rate of moisture loss and decreased the rate of contraction. However, both the rate and ultimate amount of contraction were markedly less for the various steam-cured, cement-silica mixtures than for those moist cured. These results indicate that the magnitude of stresses set up with moisture loss or gain should be considerably less in steam-cured concrete and should greatly reduce cracking or crazing of the concrete.

Thermal Expansion or Contraction. The coefficients of linear thermal expansion for steam-cured cement pastes, with and without silica, both wet and dry, for the range 70 to 140°F. averaged about 0.0000065 with maximum and minimum values about 10 per cent above and below the average. This value is practically that for moist-cured neat cement. The thermal coefficient for steam-cured concrete made with different aggregates was as follows for 1:3 mixes by volume: calcareous gravel 0.0000054, Haydite 0.0000045, granite 0.0000042, and marble, 0.0000020. It appears that the thermal expansion or contraction of steam-cured concrete depends mainly on the amount and type of aggregate in the mix.

Resistance to Sulfate Solutions. Such data as are available thus far for the first 9 months of immersion indicate excellent resistance of steam-cured concrete to 2 per cent solutions of sodium and magnesium sulfate both with and without silica. No outstanding difference between concrete steamed at high or low temperatures is yet discernible.

Resistance to Freezing and Thawing. In general, the steam-cured concrete has shown excellent resistance to freezing and thawing, particularly with regard to the exposed surfaces. The concrete steamed at 300 or 350°F. containing silica in amounts yielding optimum strength was superior to that without silica as indicated by the condition of the surfaces, corners, and edges of the cube specimens after 40 to 50 freezing and thawing cycles. It was also superior to that steamed at low temperatures either with or without silica.

CONCLUSIONS

(1) By curing in high pressure steam and proper selection of aggregates and mixtures, concrete can be obtained within 1 or 2 days after molding which will have substantially greater strength and lower volume change than concrete moist cured for 28 days at normal temperatures.

(2) Both the strength and volume change of steam-cured concrete can be controlled within wide limits by the presence or absence of finely divided silica in the cement paste and by the temperature and duration of the effective steaming period.

(3) In the absence of finely divided silica, strengths equal to the 28-day, moist-cured strength were never attained by any steaming treatment that could be applied. When steamed at 350°F. the strengths varied from about 40 to 60 per cent of that of moist-cured concrete. These relatively low strengths were accompanied, however, by lower volume changes, the contraction from saturated to dry condition ranging from about 15 to 25 per cent of that of moist-cured concrete depending on the cement used.

(4) When the cement paste contained 30 to 40 per cent of silica of about the same fineness as cement, the strength of concrete steamed at 350°F. was frequently twice or more than that of corresponding mixes without silica, but the volume change was correspondingly higher. The volume change of the steam-cured concrete, however, even with silica present, was usually less than half of that of moist-cured concrete made with cement having average volume change properties.

(5) The presence of fine silica in amounts yielding optimum strength when steamed at 350°F. produced concrete which was generally superior (with respect to denseness of paste, leaching, efflorescence, and resistance to freezing and thawing) to that of concrete steamed at 350°F. in the absence of silica. With mortars of proper design, the resistance of concrete containing silica to crazing and surface checking was equivalent to that of concrete containing no silica. For exposures up to 9 months all of the steam-cured concrete, with or without silica, appears to be unaffected by the action of either sodium or magnesium sulphate solutions.

(6) The results indicate that the useful properties of steam-cured concrete either with or without silica can be developed during exposure for 8 hr. to saturated steam at about 350°F. With an allowance of 10 to 12 hr. for gradual heating and cooling of the concrete, the nominal length of the steaming cycle becomes about 20 hr. The age at which concrete is steamed does not appear to be an important factor.

SELECTED REFERENCES

Wig, R. J.—Effect of High Pressure Steam on the Crushing Strength of Portland Cement Mortar and Concrete; Tech. Paper 5, Bur. of Stds., September 5, 1911.

Miller, D. G.—Curing Conditions of Concrete Drain Tile; *Concrete*, June, 1924, p. 233.

Miller, D. G.—Strength and Resistance to Sulfate Waters of Concrete Cured in Water Vapor at Temperatures between 100 and 350°F.; *Proc., Am. Soc. Testing Mat.*, v. 30, Part 2, p. 636, 1930.

Miller, D. G.—Discussion of Paper by Pearson and Brickett on Studies of High Pressure Steam Curing; JOURNAL, Am. Concrete Inst., v. 4, p. 101, October, 1932; *Proc.*, v. 29, 1933.

Miller, D. G. and Manson, P. W.—Laboratory and Field Tests of Concrete Exposed to the Action of Sulfate Waters; Tech. Bull. 358, Dept. Agriculture, June, 1933.

Thorvaldson, T. and Vigfusson, V. A.—Effect of Steam Treatment of Portland Cement Mortars and Their Resistance to Sulfate Action; Eng. Jl. (Canada), March, 1928.

Thorvaldson, T. and Shelton, G. R.—Steam Curing of Portland Cement Mortars; A New Crystalline Substance; Canadian Jl., *Research*, v. 1, p. 148, July, 1929.

Thorvaldson, T., Vigfusson, V. A. and Wolochow, D.—Studies on Action of Sulfates on Portland Cement; II. Steam Curing of Portland Cement Mortar and Concrete as a Remedy for Sulfate (Alkali) Action; Canadian Jl., *Research*, v. 1, p. 359, October, 1929.

Thorvaldson, T.—Discussion of Paper by Pearson and Brickett on Studies of High Pressure Steam Curing; JOURNAL, Am. Concrete Inst., v. 4, p. 102, October, 1932; *Proc.*, v. 29, 1933.

Woodworth, P. M.—Some Tests of Concrete Masonry Units Cured with High Pressure Steam; JOURNAL, Am. Concrete Inst., v. 1, p. 504, February, 1930; v. 2, p. 222, Oct., 1930; *Proc.*, Am. Concrete Inst., v. 26, 1930; v. 27, 1931.

Allan, W. D. M.—Shrinkage Measurements of Concrete Masonry; JOURNAL, Am. Concrete Inst., v. 1, p. 699, April, 1930; v. 3, p. 177, November, 1931; *Proc.*, Am. Concrete Inst., v. 26, 1930; v. 28, 1932.

Pearson, J. C. and Brickett, E. M.—Studies of High Pressure Steam Curing; JOURNAL, Am. Concrete Inst., v. 3, p. 537, April, 1932; *Proc.*, v. 28, 1932.

APPENDIX

Test Procedure

Specimens. The strength data were obtained from 2-in. cubes; those for volume change from bars 1 in. square and 11¼ in. long with effective gage length of 10 in. between stainless steel screws cast in the ends. With neat cement or mixtures of cement and powdered materials or sand up to No. 28 sieve size the mixes were sufficiently plastic for hand molding. With mixtures of cement and aggregate graded from No. 28 to ⅜ in. the concrete was of semi-plastic consistency requiring some rodding for proper molding. The semi-plastic mixes were somewhat drier than required for fully plastic concrete and somewhat wetter than required for concrete molded by vibration or power tamping.

Cement. Most of the specimens were made with the laboratory mixture consisting of equal parts by weight of 4 commercial brands of normal portland cement purchased in Chicago. In some series the influence of type of cement was studied separately with 16 commercial cements, 5 of which were of high early strength type.

Apparatus and Steaming Treatment. The specimens were steamed in a well-insulated steel cylinder with an internal diameter of 11 in. and a height of 18 in. Water was maintained in this cylinder to a depth of 3 in. This insured saturated steam at all times. The water was heated with a gas flame controlled by a thermostat. The cylinder is provided with a removable cover which can be bolted securely in position to make the steaming chamber pressure tight. The cube and bar specimens were placed on trays which held them above the water level in the atmosphere of saturated steam. Unless otherwise mentioned, the specimens were cured 24 hours in molds before being placed in the steaming chamber.

The nominal temperature of steaming was either 250, 300, or 350°F. with corresponding steam pressures of approximately 15, 55 and 120 p. s. i. (gage). Each steaming cycle consisted of a gradual heating period (from 70°F.), a period of constant temperature and pressure at the desired value and a gradual cooling period.

The heating and cooling periods were, respectively, 3 hours each for steaming at 250°; 4 hours each for steaming at 300°; and 5 hours each for steaming at 350°F. The specimens were not allowed to cool to room temperatures within the steaming chamber as this would have entailed unnecessary delay. The temperatures of the specimens on removal were 175° for those steamed at 250°; 190° for those steamed at 300°; and 200° for those steamed at 350°F.

The nominal period in the steaming chamber was 1, 2, or 3 days. Actually, the specimens steamed at 250° were in the steaming chamber 1 hour less than this and those steamed at 350° 1 hour more than this. On the basis of the above, the actual steaming periods at the maximum temperature were as shown in the following table:

TABLE 1—STEAMING PERIODS

Total Time in Steaming Chamber	Length of Constant Temperature Period of Steaming Treatment—Hours		
	Steamed at 250°F.	Steamed at 300°F.	Steamed at 350°F.
1 day (23-25 hr.)	17	16	15
2 days (47-49 hr.)	41	40	39
3 days (71-73 hr.	65	64	63
13 hr.	—	5	3
16 hr.	—	8	6
19 hr.	—	11	9

TABLE 2—COMPOSITIONS OF FINELY DIVIDED MATERIALS USED

Lot No.	Material	Silica (SiO_2)	Iron Oxide (Fe_2O_3)	Aluminum Oxide (Al_2O_3)	Calcium Oxide (CaO)	Magnesium Oxide (MgO)	Loss on Ignition	Notes
12081	Ottawa Silica	99+						
12020	Feldspar	Albite (Plagioclase Feldspar)						
12012	Wisconsin Red Granite	Chiefly orthoclase, quartz and biotite						
12016	Magnesium Limestone	5.2	3.3		27.6	19.5	43.5	49.1% $CaCO_3$ 40.7% $MgCO_3$
12018	Georgia White Marble				52.1		42.8	93% $CaCO_3$
12019	Colorado Red Lava*	49.9	29.9		9.0	5.3	0.3	
12013	Western Haydite	58.7	32.4		0.8	2.6	0.1	
12015	Soft Coal Cinders**	40.7	44.8		6.7	0.8	2.6	97.4% Ash
11893	Flue Ash	46.1	7.6	18.7	5.3	1.1	2.1	See Note***
12017	Blast Furnace Slag	36.9	12.3		42.1	3.7	1.8	
12041	Common Glass	Refractive index 1.523 ± 0.002						

*Chiefly glass of refractive index 1.632 ± 0.002 with a few crystals of quartz and feldspar.

**Ground from ⅛-¾ in. cinder clinker.

***Soluble SiO_2, 17.72%; Free Fe, 7.75%; Na_2O, 2.56%; and K_2O, 2.08%.

The fineness of the silica and feldspar (by air elutriation) expressed as per cent finer than the given micron size follows: Silica, 37.2%, 12 mic.; 56.0%, 23 mic.; 75.6%, 34 mic.; and 86.3%, 48 mic. Feldspar, 43.8%, 12 mic.; 68.1%, 23 mic.; 90.0%, 34 mic.; and 97.2%, 48 mic.

TABLE 3—GRADING OF AGGREGATES REPRESENTED IN VARIOUS DIAGRAMS OF FIG. 6

Per Cent by Weight of Graded 0-No. 28 Material in 0-¾ in. Aggregate Mixture	Per Cent by Weight of Each Sieve Size in 0-¾ in. Aggregate Mixture							
	Tyler Standard Sieve Size							
	0-200	200-100	100-48	48-28	28-14	14-8	8-4	4-¾ in.
For Calcareous Gravel								
0	0	0	0	0	20	20	20	40
10	2	2	2	4	15	15	20	40
20	4	4	4	8	10	10	20	40
30	6	6	6	12	5	5	20	40
For Granite, Limestone, Marble, Haydite and Cinders								
0	0	0	0	0	25	25	20	30
12.5	2.5	2.5	2.5	5.0	18.75	18.75	20	30
25.0	5.0	5.0	5.0	10.0	12.5	12.5	20	30
37.5	7.5	7.5	7.5	15.0	6.25	6.25	20	30

Weights per cu. ft. dry rodded: Limestone, Marble, Gravel, Granite, 125 lb.; Cinders, 80 lb.; Haydite, 56 lb.

Strength Tests. All moist-cured cubes were tested in the wet condition as taken from the moist room. All steam-cured cubes were tested wet at 70°F. after cooling in water for 2 hours after removal from the steaming chamber. In general, the strength values were based on the average strength of 3 cube specimens.

Volume Change Measurements. The volume change of all bars whether moist or steam-cured was based on their contraction in drying from the saturated condition to equilibrium with air at 70°F. and 35 per cent relative humidity. The moist-cured bars were taken directly from the foggy atmosphere of the moist room where they had been for 7 or 28 days, and exposed to the air at 35 per cent humidity. The steam-cured bars were immersed in water for 28 days before drying. All bars were stored on shelves of wire mesh with all surfaces exposed to the freely circulating air of the drying room. In general, the contractions were based on the average of 2 bar specimens.

Durability Tests. These were made by exposing 2-in. cube specimens to alternate freezing and thawing and to the action of both sodium and magnesium sulfate solutions.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for May-June 1935. Discussion should be available to the Secretary by April 1, 1935.*

Concrete in Factory Construction*

BY L. F. FAIRCHILD†

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The practice followed in the construction and maintenance of concrete structures in the several factories of the Eastman Kodak Co., at Rochester, New York, may be of value in factory construction and maintenance elsewhere.

Concrete was first used in the Eastman plant 45 years ago and since then has come into such general use in building construction that it is difficult to conceive how we would have carried on our development without it.

Of about 190,000 cu. yds. of concrete used, 172,000 cu. yds. are included in buildings having a floor area of about 4,700,000 sq. ft. The remaining 18,000 cu. yds. has gone into such supplementary work, as pavements, tunnels, reservoirs, and retaining walls.

CONCRETE IN THE NINETIES

Our records of early factory construction indicate that concrete was first used in the year 1890, for ground floors. As a foundation for wood and asphalt floors it was specified that natural cement should be mixed with sand and stone in such proportions as to make a first class concrete. Concrete topping placed on ground floor slabs was specified to be made of portland cement mixed with sharp sand in the same manner as used for the best cement sidewalks.

In the year 1896, concrete was first used for building foundations with a mix of 1:3:5.

In 1897 we find ground floors specified to be built with a 3-in. base composed of 1 part of cement to 5 parts of gravel, the gravel all being that was retained on a screen having 16 meshes to the inch. A 1-in. topping was applied consisting of two parts cement to three parts sand, the sand being that part of the gravel which passed the number 16 screen. All of these early soil-bearing floors were laid in rectangular blocks, four feet wide and six feet long.

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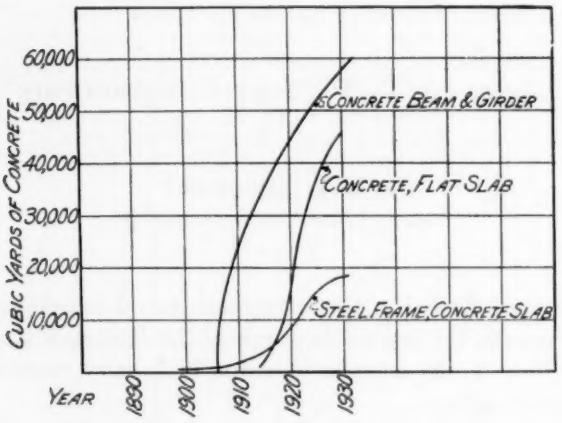

FIG. 1—RELATIVE AMOUNTS OF CONSTRUCTION OF THREE MAJOR TYPES

The first reinforced concrete slab was built in 1898, to serve as a roof, using cinders as coarse aggregate and it was not until 1905 that a reinforced floor slab was placed with the use of stone aggregate. The building of this slab was under special contract and was not included in the mason work.

The first building with a reinforced concrete frame was erected in 1906, and was built with floors and roof slabs of concrete joists, separated by tile, then known as the Kahn construction. In 1909 there appeared the first all-concrete beam and girder structure.

BUILDING FRAMES

Since concrete was first used for reinforced slab construction in 1898, 102 major buildings have been erected, using concrete in the structural frame above the foundation. Fifty-seven of these buildings, having a total floor area of 3,622,000 sq. ft. are built with concrete frames, using a total volume of concrete of about 105,000 cu. yds. Forty-five buildings, with a total floor area of 908,700 sq. ft., have steel frames, with 18,000 cu. yds. of concrete in the floor and roof slabs.

The curves of Fig. 1, indicate the relative amounts of construction of the three major types, namely, (1) steel frame and concrete slab, (2) concrete beam and girder, and (3) concrete flat slab. The construction is measured in cubic yards of concrete, exclusive of foundations. It will be noted that all three types were under construction during the 17 years previous to 1931. While the "concrete slab on steel frame" was the first to be used (1898), the amount of construction was moderate until about 1910. Since that time its use has been fairly

active, reaching a total amount of 18,000 cu. yds. During this time, however, it was far exceeded by the construction having concrete frames. The first concrete beam and girder building was constructed in 1906 and the first flat slab structure in 1914. The construction of both of these types has been very active, as indicated by the rapid rise of the curves, 60,000 cu. yds. of concrete having been used in the beam and girder type, and 45,000 cu. yds. in the flat slab.

It is unnecessary to state that economy is the most important factor in deciding what type of frame should be used in factory construction, and it has been generally agreed that flat slab is the most economical fireproof construction for factory purposes. This is borne out by the fact that since the first flat slab was built in 1914, 45,000 cu. yds. of concrete have been used in this type as compared with the 27,000 cu. yds. in the beam and girder structures.

VARIOUS CONSTRUCTION FEATURES

Profiting by experience on our early concrete buildings, some of which are now 25 years old, and assisted by the study and research that has been devoted to this subject by organizations throughout the country, we have been able to overcome difficulties which were a source of trouble in the past.

The selection of proper aggregate, the careful proportioning of materials, the use of the water cement ratio, special care in placing concrete so as to secure a dense finished product, and the use of the proper curing methods, are factors which must be considered if permanent concrete is to be obtained.

Certain construction details have been a source of trouble and maintenance has been costly. To overcome these difficulties, many improvements have been made of which some of the following are typical examples.

Keeping Steel Away From Surface

To keep the reinforcing steel away from the exterior surface of the concrete, we have adopted the practice of using precast concrete washers on the stirrups on the exterior sides of the beams. These washers definitely provide the proper distance between the stirrup and the face of the beam. (Fig. 2.) The stirrups of many buildings erected prior to the use of this method are near the exterior surface of the beams and have caused disintegration due to rusting. Several methods of repair were tried before a successful one was secured. The method now used is thoroughly to remove all loose concrete, clean the steel with a wire brush, wet the beam and then apply a cement mortar of a 1 to 3

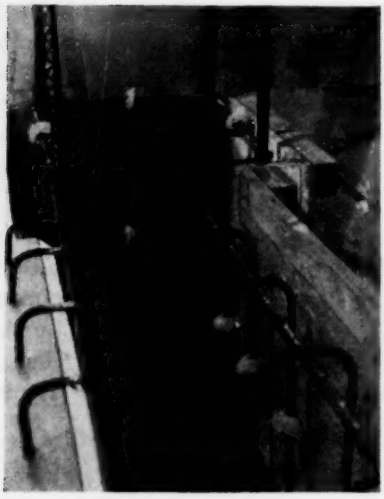

FIG. 2—CONCRETE WASHERS ARE USED TO PREVENT STIRRUPS TOUCHING THE BEAM FORMS

FIG. 3—SMALL SIPHON USED FOR APPLYING MORTAR AT CLOSE RANGE

mix, with a small siphon. The mortar is mixed to the consistency of heavy grout and placed in a pail. The lower end of a siphon is then inserted in the pail and the mortar shot onto the beam with air pressure at 80 pounds, similar to cement gun work. (Fig. 3.) These patches are cured for two days by hanging heavy wet burlap pads over the repairs. This provides dense concrete over the steel with excellent bond. Successful repairs of this nature have been made over a period of four years. (Fig. 4.)

To Prevent Rusting of Steel Sash

The method of constructing concrete window sills now generally used prevents the rusting of the bottom of the steel sash. The old and improper method of imbedding the lower fin of the steel sash in the sill has caused failure to both the sash and sill. (Fig. 5 and 6.) The sash buckles due to the pressure from the formation of rust at the sill and in some cases the sill is broken due to this same pressure. Unless corrected in its early stage, this trouble cannot be repaired; it becomes necessary to replace the sill and in many cases also the steel sash.

Exposed Walls

Climate is an important factor in the durability of concrete structures. The winters of northern New York are long and often severe and exposed concrete surfaces must be of good quality. Concrete

FIG. 4—STIRRUPS IN ALL BEAMS ON THIS SIDE OF THE BUILDING WERE REPAIRED

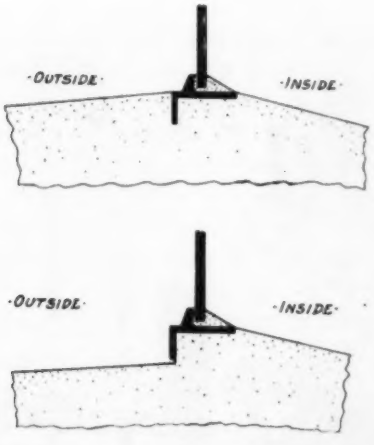

FIG. 5 AND 6—CONCRETE WINDOW SILL WITH ROLLED STEEL SASH—OLD DETAIL WHICH HAS BEEN DISCONTINUED (TOP) NEW 'STANDARD' PRACTICE (BOTTOM)

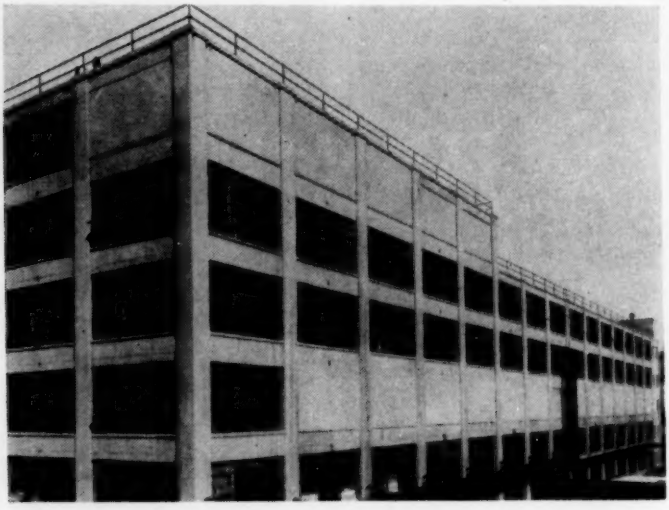

FIG. 7—PILASTERS STOPPED AT ROOF LEVEL

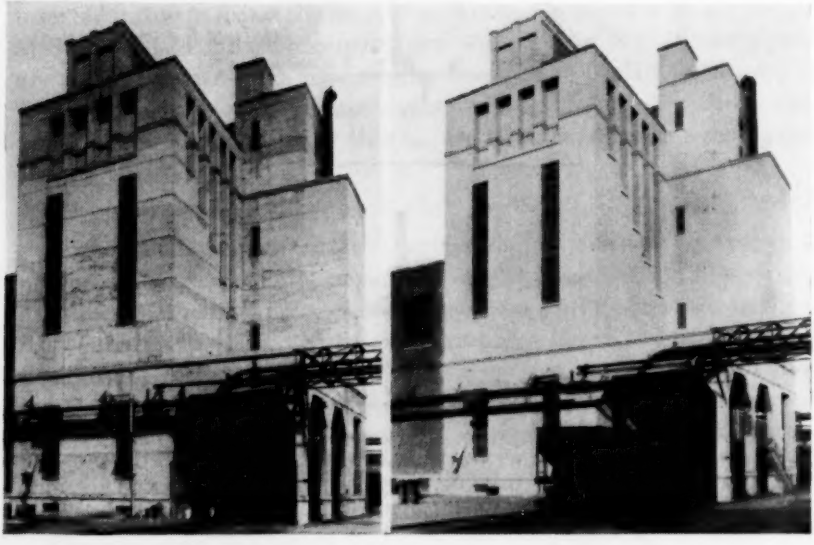

FIG. 8—BUILDING 13 YEARS OLD, UNPAINTED
FIG. 8a—SAME BUILDING AFTER BEING PAINTED

must be dense to prevent absorption and exposed horizontal surfaces should be avoided wherever possible. Where exposed walls are necessary, the top surfaces should be sloped and given a dense, troweled surface to prevent absorption from standing water. On our more recent buildings pilasters have been terminated at the roof level. (Fig. 7.) This practice eliminates the maintenance work which has been found necessary on some of the older buildings where the parapet walls and pilasters extending above the roof level have disintegrated.

Painting

It is our standard practice to paint exposed concrete surfaces of all buildings for the sake of appearance and with the belief that it helps to prevent surface disintegration. One building, however, was 13 years old before it was painted, and although somewhat discolored, did not show serious indications of trouble. (Fig. 8 and 8A.)

In considering such repairs as have been mentioned it must be realized that the amount of defective concrete is relatively small and has been due to the lack of knowledge or to carelessness in making or placing the material.

High Early Strength Cement

High early strength cement has been used extensively in this plant for several years for underpinning old structures and for other work where full loads have been applied 24 hours after placing. This concrete has given every indication of being satisfactory after a period of several years. In constructing a new basement under an existing power house, the entire building, covering an area of 14,000 sq. ft., was shored and new basement columns supporting the building and the boilers were built with the use of this cement. (Fig. 9.) Full building load, amounting to about 1,000,000 lb. per column, was applied 24 hours after placing. All concrete made with this cement was mixed and placed strictly in accordance with the instructions of the manufacturer.

Expansion Joint Repairs

Expansion joint repairs sometimes present a difficult problem. Repairs have been necessary in one 5-story building, erected in 1917, where heavy girders rest on brackets. Due to the absence of steel to steel bearing, these girders failed to slide as was intended. Failure occurred by shearing either in the bracket or in the beam. (Fig. 10.) Repairs were made by securing steel brackets to the column, or girders, with cinch anchors, and placing steel channels on the bottom of the beams so as to provide the proper steel to steel contact for sliding. (Fig. 11.) Forty brackets have been repaired in this manner in one

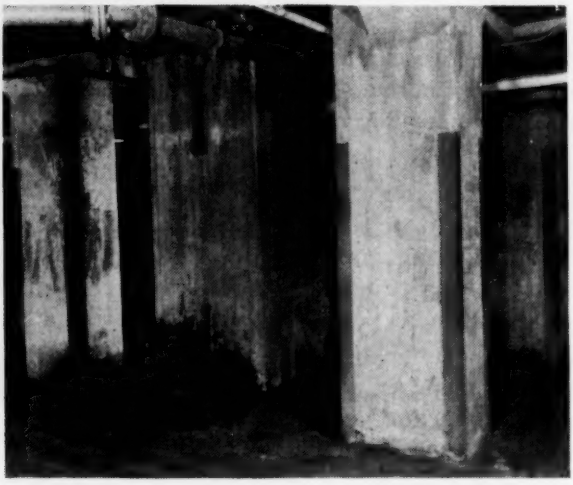

FIG. 9—IN CONSTRUCTING BASEMENT UNDER EXISTING POWER HOUSE, 16 COLUMNS SUCH AS THESE WERE BUILT WITH THE USE OF HIGH EARLY STRENGTH CEMENT

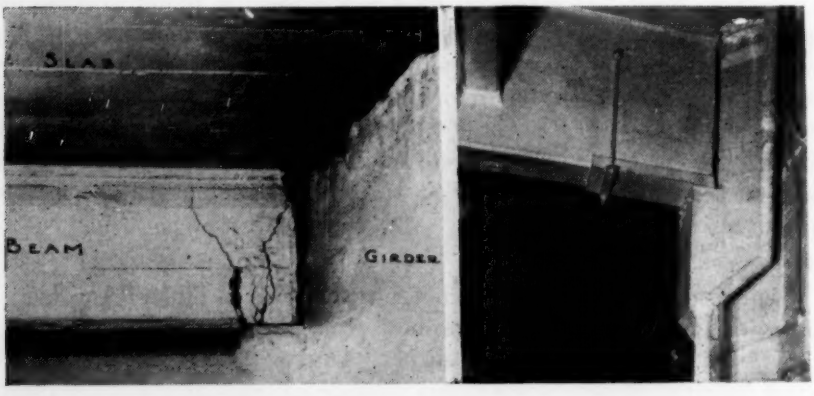

FIG. 10 AND 11—BRACKET AND BEAM AT EXPANSION JOINT BEFORE AND AFTER REPAIRS

FIG. 12—CONCRETE ALONG THE BOTTOM OF THE BEAMS WAS EASILY REMOVED

building without giving any further trouble. It has been generally agreed that the most satisfactory way of dealing with an expansion joint in a new building is to construct independent columns spaced two inches apart. This, in reality, means two separate structures, although both columns at the joint may rest on the same foundation.

Faulty Beam Bottoms

A few years ago we found in one of our buildings that small sections of concrete had fallen from the bottom of the beams in several places. Investigation showed the concrete in the bottom of such beams below the horizontal reinforcement to be of poor quality due apparently to improper mixing or placing. In many cases a layer, about one inch thick below the reinforcement, containing no coarse aggregate, and was easily removed for the full length of the beam (Fig. 12), disclosing a definite horizontal cleavage plane. Satisfactory repairs were made by securing expanded metal lath to the reinforcing steel and plastering with cement mortar.

Floor Finish

In factory buildings, floor finish is probably the most difficult item to maintain. To secure definite data of our own on this subject, we conducted a series of tests, using various aggregates in different proportions. The testing machine was so constructed that loaded trucks traveled in a circular path over the test area. (Fig. 13.) Four series

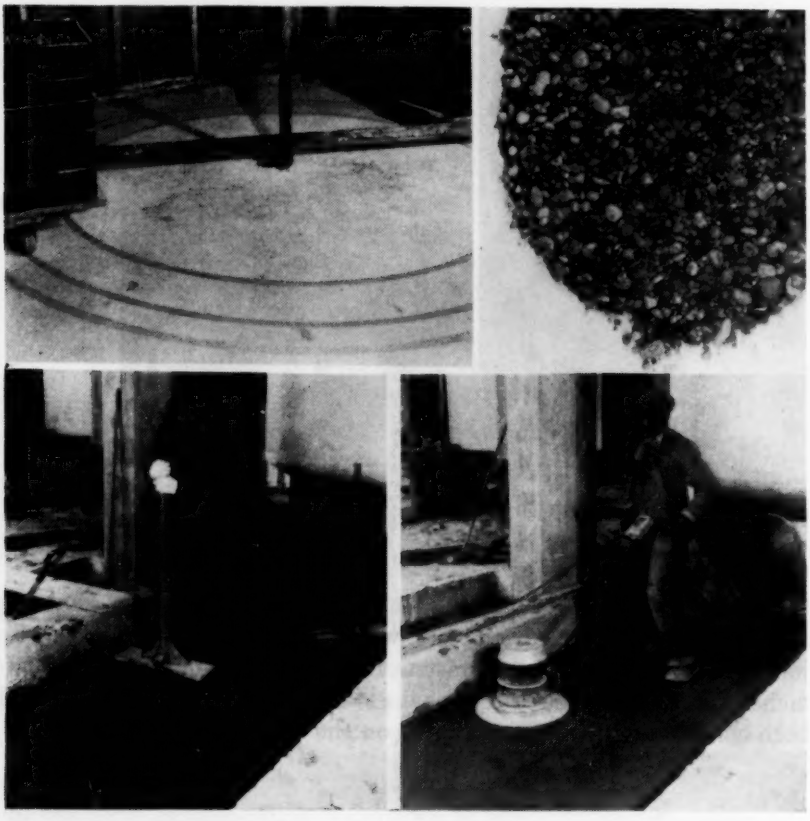

FIG. 13—MACHINE FOR TESTING FLOOR FINISH
FIG. 14—SILICA SAND AGGREGATE USED FOR FLOOR FINISH
FIG. 15—TAMPING OF FINISH MATERIAL
FIG. 16—APPLICATION OF POWER TROWEL

of tests were made, using aggregates such as ordinary sand, iron ore tailings, crushed quartz, and pure silica sand. These were tested with varying fineness moduli and varying strengths. One of these tests, where silica sand was used, is of special interest. (Fig. 13a and 13b.) Curves for three fineness moduli are shown on Fig. 13a and F. M. 3.93 indicates less wear than either the coarser or finer grades. Curves for the three concrete strengths are shown on Fig. 13b indicating increased resistance to wear with increased strength. The best combination therefore would be F. M. 3.93 and 5,500 lb. strength. The results of these tests indicated the advisability of adopting the following practice:

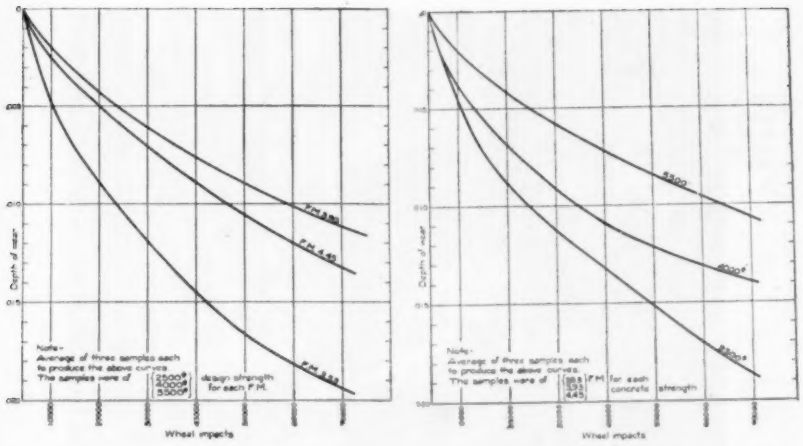

FIG. 13a—AVERAGE CURVES FOR FINENESS MODULI FLOOR FINISH
FIG. 13b—AVERAGE CURVES FOR CONCRETE STRENGTH FLOOR FINISH

Silica sand with a fineness modulus of about 4 (Fig. 14), uniformly graded with sizes ranging from those that pass a ⅜-in. mesh to 97 per cent retained on a number 48 mesh. 3.1 parts of this sand are mixed with one part of cement, using 3.5 gallons of water per bag—this for 5,500 p. s. i. This material is mixed for not less than three minutes. After the material is spread, thoroughly tamped (Fig. 15), and screeded, it is treated with a power trowel. (Fig. 16.) It is then immediately given one or two treatments of the hand trowel according to the degree of smoothness required, and the completed finish is kept wet for not less than 10 days. This floor finish material in the above proportions is so dry that it cannot be finished with hand troweling alone, but with the use of a power trowel, just enough paste is brought to the surface to furnish an excellent finish. This troweling does not bring up any excess fine material nor any free water. The silica sand remains at the surface with a minimum amount of paste holding the sand particles together, and the surface wear is taken by the sand rather than by the cement. It seems reasonable, therefore, that surface hardeners would be of little value to this finish because the cement, which is the ingredient affected by the hardeners, takes little, if any of the wear.

For Waterproof Floor Finish

Where waterproof conditions are desired, a 2 course finish, 2½ in. thick, has been developed to rest on a waterproof membrane. This finish is divided into sections about 10 ft. square or smaller, and these sections are separated from each other by joints ¼ in. wide, formed by

FIG. 17—ARTIFICIAL STONE BASE—ROCK-FACED AFTER CASTING

a special tool, and then filled with a compound after the finish has hardened. To economize material, the lower layer, 1¾ in. thick, is made of one part cement, one part standard sand, and two parts pea gravel. Steel reinforcement is imbedded in this lower layer. The upper layer, ¾ in. thick, is the silica sand finish of proportions as described above, and is laid monolithic with the lower layer.

Foundations

The foundations of all important structures are carried to bed rock which is from 10 to 20 feet below ground level. We have found that with a six-story building, it costs approximately the same to go 14 ft. to rock or to go 4 ft. to soil bearing with spread fottings. The spread footing would be about 12 ft. square, and the pier to rock about 3 ft. square. These sizes are based on the bearing values required by the Rochester Building Code. With the cost the same, the rock bearing is of course much more desirable.

Roof Slabs

It is our present practice with all concrete buildings to build level roof slabs designed for a live load of 150 lb. per sq. ft. This permits the construction of unforeseen roof additions which are often found very desirable after the building is completed.

Precast Concrete Trim

In some of the more recent buildings, precast concrete trim has been used. In one building alone, a saving of $30,000 was realized by using

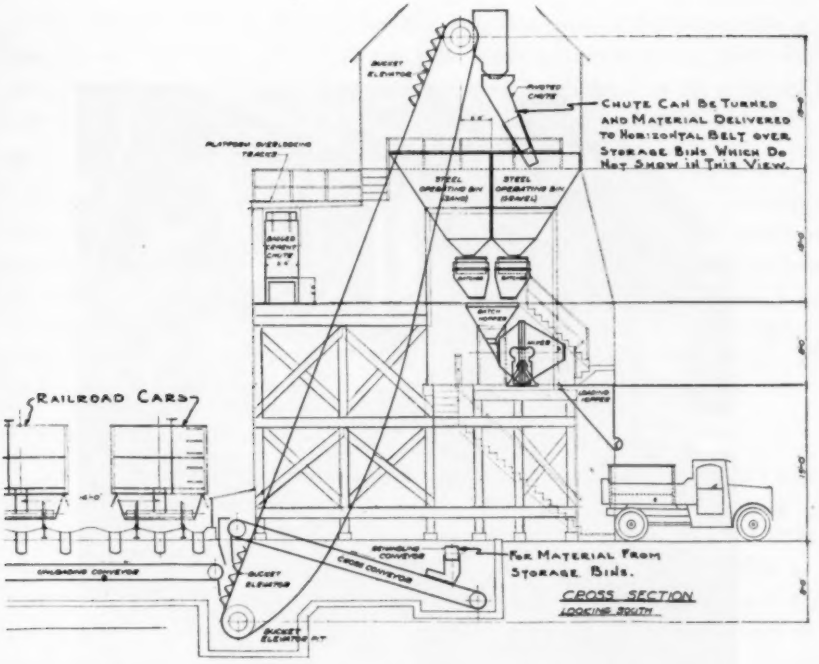
CHUTE CAN BE TURNED AND MATERIAL DELIVERED TO HORIZONTAL BELT OVER STORAGE BINS WHICH DO NOT SHOW IN THIS VIEW.
RAILROAD CARS
FOR MATERIAL FROM STORAGE BINS.
CROSS SECTION
LOOKING SOUTH

FIG. 18—CROSS SECTION OF CENTRAL MIXING PLANT

this material in place of sandstone. (Fig. 17.) On other buildings, both poured in place and precast belt courses and window sills have been built of colored concrete to match stone trim on adjacent buildings. Some of this trim was rock faced *after* being cast.

Central Mixing Plant

Owing to the volume of concrete construction started in 1930, and the saving which would be realized, a central concrete mixing plant was constructed. The entire plant, including the sand and stone storage bins, is completely enclosed. An underground tunnel conveys material from the track hoppers to the mixing hoppers or to the storage bins by belt and bucket conveyors. Storage bins hold material for 600 cu. yds. of concrete and are heated by perforated steam lines placed at the bottom of the bins at 4 ft. centers. The plant is equipped with two tilting mixers, each having its own batcher, and operating bin above. (Fig. 18.) The mixing platform is 15 feet above grade and the mixed concrete is dumped directly into trucks. (Fig. 19.) For transporting the concrete, removable steel tubs of half cylinder cross section are secured temporarily to standard trucks. Concrete mixed

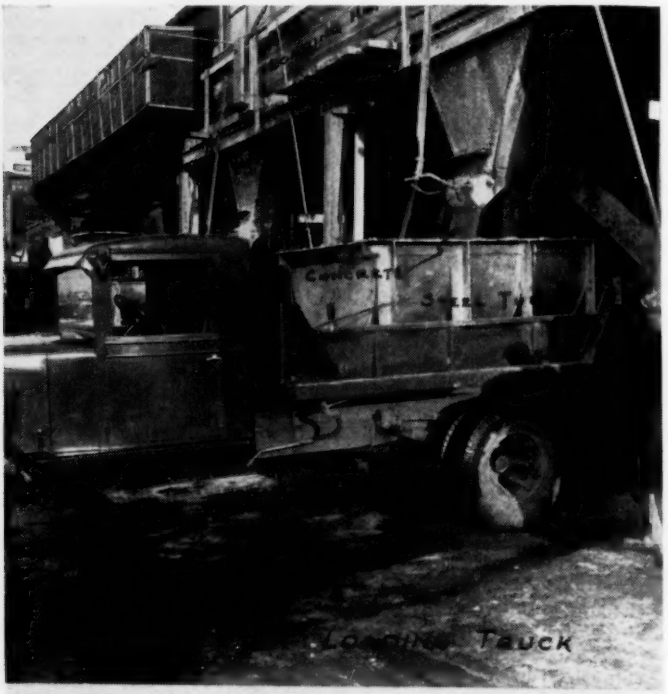

FIG. 19—CONCRETE TRUCK UNDER LOADING HOPPER

FIG. 20—CONCRETE POLES

at this plant (with an admixture) has been transported successfully on long hauls. On one job, 1500 cu. yds. were trucked seven miles and placed without difficulty. On another job, the haul was 17 miles and here the concrete was placed easily with the use of a vibrator.

Miscellaneous Uses of Concrete

Throughout the plant, concrete has been used extensively for bridge abutments and retaining walls. Seventeen thousand square yards of concrete pavements have been built, together with many concrete sidewalks. Reinforced concrete tunnels have been constructed to connect various buildings. As a part of the water supply system, a pumping station and filter beds have been constructed of concrete, as well as a 5,000,000 gallon storage reservoir.

Concrete Poles

When it became necessary to make an extensive run of various pipe lines in a new territory, it was decided to construct supporting poles of concrete. (Fig. 20.) There are 65 of these poles, 20 ft. high, and 20 ft. on centers, shot in place with gunite. The portions below grade are concrete cast in place. Above grade the forms in the shape of three sided boxes were erected, in a vertical position, the reinforcement installed, and the forms then filled with concrete with the use of the cement gun. These poles were built of 4,000 lb. concrete with a six foot cross arm at the top, designed for a 2,000 lb. load at each end. One pole was tested with double this load at one end of the arm and although the pole was deflected 9½ in. out of the vertical, it returned to its original position when the load was removed. These poles were built in 1929 and are in perfect condition today.

Tanks

Since 1920, considerable economy has been realized in the construction of tanks made of gunite in place of wood. In one building, 177 of these tanks were constructed at a saving of $28,000 from the figure for wooden tanks.

Fig. 21 shows a detail of these tanks, used as liming vats in the gelatin plant, about 7 ft. 6 in. x 15 ft. 10 in. and 3 ft. 6 in. deep. The floor is 4 in. thick of poured concrete, reinforced horizontally and with reinforcement projecting vertically into the walls which are designed as cantilevers from the floor. All walls are 2 in. thick at the top. The exterior walls are 2½ in. thick at the base and the interior walls which receive pressure from either side are 3 in. thick. The mix used for the gunite was one part portland cement and three parts screened sand—designed for a working stress of 1200 p. s. i. The tanks cost $0.97

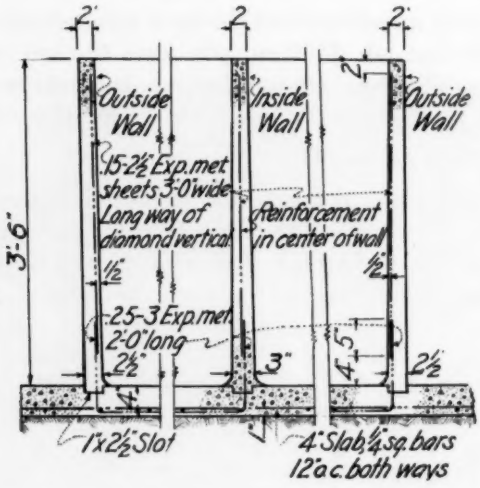

FIG. 21—TYPICAL CROSS-SECTION OF CONCRETE TANKS

per sq. ft. of floor area, whereas the cost of cypress tanks would have been $1.89. The 177 tanks involved about 480 cubic yards of concrete.

Recently open cylindrical tanks, 7 ft. in diameter, and 10 ft. high, with walls 3 in. thick, have been built of gunite and have proven very successful.

It will be evident that concrete is serving a variety of purposes in our Kodak factory construction. Difficulties are being corrected, the art is steadily improving and a wide field is open for its future development.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for May-June 1935. Discussion should be available to the Secretary by April 1, 1935.*

Further Studies of Temperature Effects on Compressive Strength of Concrete*

BY A. G. TIMMS† AND N. H. WITHEY‡

SUPPLEMENTING the authors' paper, "Temperature Effects on Compressive Strength of Concrete," JOURNAL of the American Concrete Institute, January-February, 1934,[1] studies were made of additional phases of the subject. In the earlier investigation the concrete upon removal from the cold rooms was stored in air at 70 ± 6° F. and at about 50 per cent relative humidity and no moisture was furnished for further curing. In the present series the effect of subsequent moist curing in developing the potential strength of the concrete was studied. This investigation provides, in addition, information on the effect on compressive strength of storing concrete continuously in water at 33 ± 2° and 50 ± 2° F.

MATERIALS

Sand and gravel aggregates from Elgin, Ill. were oven dried and screened to the following sizes: 0-No. 28; No. 28-14; No. 14-8; No. 8-4; No. 4-⅜ in.; and ⅜-¾ in. These six sizes were recombined for each batch to give a constant grading the same as that used in the previous investigation.

One high-early strength cement and laboratory mixture (a mixture of equal parts of four brands of normal portland cement) were used. The cements were the same brands as those used for the laboratory mixture and high-early strength cement No. 1 in the previous investigation, but were from different shipments and gave slightly different age-strength relationships. The water-stored specimens were made with a different lot of high-early strength cement than that used in the air-stored specimens, but the same laboratory mixture was used throughout.

*Received by the Secretary Nov. 5, 1934.

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[1]JOURNAL Amer. Concrete Inst., *Proceedings* Vol. 30, p. 159. (See also discussion Sept.-Oct., 1934, p. 9 and this issue p. 197.)

The mix used was 1:2.78:3.33 by weight for the laboratory mixture and 1:2.52:3.03 for the high-early strength cement. These are the same mixes used for the corresponding water content of 6 gal. per sack of cement in the earlier tests.

PROCEDURE FOR MAKING AND STORING SPECIMENS

Batches sufficient in size to make 8 to 16 test cylinders, depending upon the number required for the different storage conditions, were mixed for 2 minutes in Lancirick open-tub mixers of ½ or 1¼ cu. ft. capacity. In general, four specimens were made for each test condition; one each on four different days. The molds for the specimens given water curing at 72 ± 6° F. following exposure, were cylindrical cardboard tubes 3 in. in diameter by 6 3/16 in. long, paraffin treated. These tubes were fitted over machined cast iron base plates 3/16-in. thick to give a specimen 6 in. high when finished flush with the top of the tube. The molds were sealed to the plate with a mixture of paraffin and rosin, and supported in frames in gangs of 8 to 10. Filling was done in 3 layers, each layer being rodded 25 times with a ¼-in. bullet-pointed steel rod.

Wherever possible, the tops of the cylinders were capped with neat cement 3 to 5 hr. after making, in accordance with A. S. T. M. Standard Methods. When the curing condition made it impossible to cap with neat cement, neat gypsum plaster was used just before testing. The bottoms of the cylinders were sufficiently plane without capping. When the concrete was to be placed in water immediately or 6 hr. after molding, cast iron cylinders with sealed bases were used, and sheet metal discs were sealed to the tops.

Specimens to be exposed to low temperatures were first given preliminary curing in air (in the molds but not covered) before placing in the cold rooms. The temperature during this preliminary curing was the same as that of the concrete at time of placing; that is, either 80 ± 6° F. or 50 ± 4° F. For the specimens exposed in air, preliminary curing periods of ¼ and 1 day were used. For the specimens stored in water, 4 preliminary curing periods were used: 0 day, ¼ day, 1 day and 3 days.

Specimens exposed in air to temperatures of 16 and 33° F. and given subsequent warming treatments were tested at ages of 1, 3, 7, 28 and 90 days. When the specimens were stored in water, 3 storage temperatures were used: 33 ± 2, 50 ± 2, and 72 ± 6° F. These specimens were tested at ages of 1, 3, 7, and 28 days.

The various combinations of storage conditions can best be ascertained from Tables 1 to 4.

TABLE 1—COMPRESSIVE STRENGTH OF CONCRETE EXPOSED TO LOW TEMPERATURES IN AIR AND SUBSEQUENTLY WARMED UNDER WET AND DRY CONDITIONS

Test specimens 3 by 6-in. concrete cylinders. Specimens stripped of molds upon removal from cold room. Specimens soaked in water at 72° F. for 2 to 3 hr. before testing. Concrete placed at 80° F. Each value is the average of tests on 4 to 10 specimens.

Period of Exposure to Low Temperature in the Molds—Days	In Water at 72° F. After Exposure—Days	Compressive Strength—lb. per sq. in.									
		Lab. Mixture (Lot 12000)					High Early Strength Cement (Lot 12248)				
		1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.
		Moist Cured Entire Time at 75° F.									
		830	2860	4220	6030	7070	1690	4840	4960	6790	7580
¼ Day Preliminary Storage in Molds in Air at 80° F. before Exposure at 33° F.											
2¾	0		700	2380	2860	2570		1680	4680	4890	4740
	3			3350	4650	4370			5620	6380	6140
	4			3350	——	——			5440	——	——
	7				5610	5360				6670	5970
	25				5500	——				6510	——
	28					6270					6670
	87					6600					7290
6¾	0			1650	3450	3220			3760	5350	4790
	3				4920	4910				6380	5730
	7				5340	5390				6410	6240
	21				5410	——				6550	——
	28					6670					7000
	83					6520					7090
27¾	0				3020	3480				4540	5190
	3					5060					6100
	7					5450					6410
	21					6000					6890
	62					6050					7120
1 Day Preliminary Storage in Molds, in Air at 80° F. before Exposure at 33° F.											
2	0		2150	3300	3850	3310		3780	5040	5500	5090
	3			3880	5220	4690			5830	6620	5900
	4			3790	——	——			5520	——	——
	7				5880	5450				6800	6290
	25				5470	——				6480	——
	28					6400					7090
	87					6910					7180
6	0			2380	3950	3640			4560	5370	5150
	3				5330	5170				6240	6090
	7				5770	5480				6770	6200
	21				5490	——				6190	——
	28					6620					7250
	83					6610					6720
27	0				3230	3940				5280	5380
	3					5190					6060
	7					5580					6360
	21					6240					6960
	62					6320					6760
¼ Day Preliminary Storage in Molds in Air at 80° F. before Exposure at 16° F.											
2¾	0		140	1820	2310	2180		310	3380	4010	3830
	3			2830	4140	3960			5540	6410	5990
	4			2790	——	——			5170	——	——
	7				5000	4930				6980	6800
	25				5440	——				6920	——
	28					5990					7500
	87					6580					7200
6¾	0			210	1810	2190			360	3360	3600
	3				3820	4240				6320	6610
	7				4960	4950				6720	6850
	21				5460	——				6840	——
	28					5710					7040
	83					6900					7610
27¾	0				270	2220				460	3250
	3					3950					6030
	7					4630					6570
	21					6180					7070
	62					6510					7120

(Table 1 continued on page 168)

TABLE 1 (Continued)—COMPRESSIVE STRENGTH OF CONCRETE EXPOSED TO LOW TEMPERATURES IN AIR AND SUBSEQUENTLY WARMED UNDER WET AND DRY CONDITIONS

Test specimens 3 by 6-in. concrete cylinders. Specimens stripped of molds upon removal from cold room. Specimens soaked in water at 72°F. for 2 to 3 hr. before testing. Concrete placed at 80°F. Each value is the average of tests on 4 to 10 specimens.

Period of Exposure to Low Temperature in the Molds—Days	In Water at 72° F. After Exposure—Days	Compressive Strength—lb. per sq. in.									
		Lab. Mixture (Lot 12000)					High Early Strength Cement (Lot 12248)				
		1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.
1 Day Preliminary Storage in Molds in Air at 80° F. before Exposure at 16° F.											
2	0		1210	2940	3560	3210		3000	4760	5140	4910
	3			3670	5160	5040			5570	6180	6460
	4			3650	——	——			5410	——	——
	7				5750	5670				6600	6510
	25				6060	——				6640	——
	28					6880					7480
	87					7040					7520
6	0			1330	3230	3360			3170	5120	4640
	3				4880	5110				6440	6280
	7				5560	5410				6700	6350
	21				5790	——				6490	——
	28					7010					7390
	83					6970					7200
27	0				1540	3380				3260	4790
	3					4750					6540
	7					5700					7050
	21					6780					7330
	62					6770					7140

Comparison or "key" specimens were made in the same manner and given moist curing entire time at 75° F. (molds removed at 24 hr.)

Concrete was placed at 80 or 50° F. as shown in the tables. When placed at 50° F. cement, aggregates, and water were pre-cooled before mixing. As in the earlier tests, the temperature of the materials before mixing was so adjusted that the warming effect of mixing and chemical action produced the desired temperature at the time of placing.

Before testing, all specimens were soaked 2 to 3 hr. in water at 72° F., which constituted the only warming for those specimens tested immediately upon removal from the cold room. While this period in water was sufficient to warm the specimens to normal temperature it was not sufficient to bring them to the same degree of saturation. This probably affected some of the strength test results.

PRINCIPAL RESULTS OF THE TESTS

Complete strength data for these tests are given in Tables 1 and 3. Percentage values derived from the strengths are given in Tables 2 and 4. The data are also shown in Fig. 1. to 9.

RESULTS FOR NORMAL PORTLAND CEMENT

Fig. 1 shows age-strength relationships for concrete made at 80° F. with laboratory mixture. The upper two rows of diagrams are for

TABLE 2—DATA FROM TABLE 1 EXPRESSED AS A PERCENTAGE OF THE 28-DAY STRENGTH OF MOIST CURED CONCRETE

Period of Exposure to Low Temperature in the Molds—Days	In Water at 72° F. After Exposure—Days	Compressive Strength—lb. per sq. in.									
		Lab. Mixture (Lot 12000)					High Early Strength Cement (Lot 12248)				
		1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.
		Moist Cured Entire Time at 75° F.									
		14	48	70	100	117	24	70	86	100*	109
¼ Day Preliminary Storage in Molds in Air at 80° F. before Exposure at 33° F.											
2¾	0		12	40	47	43		24	68	71	68
	3			56	77	72			81	92	89
	4			56	—	—			79	—	—
	7				93	89				96	86
	25				91	—				94	—
	28					104					96
	87					110					105
6¾	0			27	57	53			54	77	69
	3				82	81				92	83
	7				89	89				93	90
	21				90	—				95	—
	28					111					101
	83					108					102
27¾	0				50	58				66	75
	3					84					88
	7					90					93
	21					99					99
	62					100					103
1 Day Preliminary Storage in Molds in Air at 80° F. before Exposure at 33° F.											
2	0		36	55	64	55		55	73	79	73
	3			64	87	78			84	96	85
	4			63	—	—			80	—	—
	7				97	90				98	91
	25				91	—				94	—
	28					106					102
	87					115					104
6	0			40	66	60			66	77	74
	3				89	86				90	88
	7				96	91				98	90
	21				91	—				89	—
	28					110					105
	83					110					97
27	0				54	65				76	78
	3					86					87
	7					93					92
	21					104					100
	62					105					98
¼ Day Preliminary Storage in Molds in Air at 80° F. before Exposure at 16° F.											
2¾	0		2	30	38	36		4	49	58	55
	3			47	69	66			80	93	86
	4			46	—	—			75	—	—
	7				83	82				101	98
	25				90	—				100	—
	28					99					108
	87					109					104
6¾	0			3	30	36			5	49	52
	3				63	70				91	96
	7				82	82				97	99
	21				90	—				99	—
	28					95					102
	83					115					110
27¾	0				5	37				7	47
	3					66					87
	7					77					95
	21					102					102
	62					108					103

*This value is taken from the curve in Fig. 2 and is used as the basis for all the other values for high-early-strength cement.

(Table 2 continued on page 170)

TABLE 2 (Continued)—DATA FROM TABLE 1 EXPRESSED AS A PERCENTAGE OF THE 28-DAY STRENGTH OF MOIST CURED CONCRETE

Period of Exposure to Low Temperature in the Molds—Days	In Water at 72° F. After Exposure—Days	Compressive Strength—lb. per sq. in.									
		Lab. Mixture (Lot 12000)					High Early Strength Cement (Lot 12248)				
		1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.
1 Day Preliminary Storage in Molds in Air at 80° F. before Exposure at 16° F.											
2	0		20	49	59	53		43	69	74	71
	3			61	85	84			80	89	93
	4			61	—	—			78	—	—
	7				95	94				95	94
	25				100	—				96	—
	28					114					108
	87					117					109
6	0			22	54	56			46	74	67
	3				81	85				93	91
	7				92	90				97	92
	21				96	—				94	—
	28					116					107
	83					116					104
27	0				25	56				47	69
	3					79					94
	7					95					102
	21					112					106
	62					112					103

TABLE 3—COMPRESSIVE STRENGTH OF CONCRETE EXPOSED TO DIFFERENT TEMPERATURES IN WATER

Test Specimens 3 by 6-in. concrete cylinders.
Specimens given 0 or ¼ Day preliminary storage sealed in cast iron molds before exposure in water and stripped of molds at 1 day.
All other specimens made in cardboard molds and stripped just prior to exposure in water.
Each value is average of from 4 to 8 tests.
One day specimens soaked in water at 72° F. for 1 hr., all others 2 to 3 hr. before testing.

Temp. of Concrete When Placed °F.	Preliminary Storage in Molds in Air at Placing Temp., Days	Temp. of Exposure in Water °F.	Compressive Strength—lb. per sq. in.							
			Laboratory Cement Lot 12000				High Early Strength Cement Lot 12300			
			1d.	3d	7d.	28d.	1d.	3d.	7d.	28d.
80	0	72	650	1890	3000	4520	1670	3600	4320	5170
	¼		830	2420	3830	5580	2210	4590	5480	6180
	1		800	2670	3860	5910	1990	4860	5960	7020
	3			2530	4050	5700		4600	5670	6830
80	0	50	230	990	2180	3800	810	2920	4170	5210
	¼		480	1440	2840	4810	1450	3720	4960	6480
	1		800	2120	3040	4970	1990	4770	5410	6870
	3			2530	3590	4930		4600	5680	6710
50	1	50	180	750	2210	4550	720	2680	4940	6430
80	0	33	30	270	1100	2720	260	1540	3380	4790
	¼		250	950	1850	3790	1000	2820	4310	5340
	1		800	1630	2550	3980	1990	3750	4880	5650
	3			2530	3310	4410		4600	4760	5460
50	1	33	180	590	1550	3770	720	2220	4360	5670

TABLE 4—DATA FROM TABLE 3 EXPRESSED AS A PERCENTAGE OF THE 28-DAY STRENGTH OF CONCRETE CURED 1 DAY MOLDS, THEN IN WATER AT 72° F.

Temp. of Concrete When Placed °F.	Preliminary Storage in Molds in Air at Placing Temp., Days	Temp. of Exposure in Water °F.	Per Cent of 28-Day Strength of Concrete Cured 1 Day Molds, then in Water at 72° F.							
			Laboratory Cement Lot 12000				High Early Strength Cement Lot 12300			
			1d.	3d.	7d.	28d.	1d.	3d.	7d.	28d.
80	0	72	11	32	51	76	24	51	62	74
	¼		14	41	65	94	31	65	78	88
	1		14	45	65	100	28	69	85	100
	3			43	68	96		65	81	97
80	0	50	4	17	37	65	12	42	59	74
	¼		8	24	48	81	21	53	71	92
	1		14	36	51	84	28	68	77	98
	3			43	61	83		65	81	96
50	1	50	3	13	37	77	10	38	70	92
80	0	33	1	5	19	46	4	22	48	68
	¼		4	16	31	64	14	40	61	76
	1		14	28	43	67	28	53	69	80
	3			43	56	75		65	68	78
50	1	33	3	10	26	64	10	32	62	81

concretes exposed at 33° F. and the bottom two rows for concrete exposed at 16° F. It should be noted that the curve representing continuous moist curing is the same in all diagrams. The curves representing continuous exposure at 33 and 16° F. are the same in any horizontal row. The other curves represent concrete stored in warm water or in warm air or combinations of both after removal from the cold rooms.

In studying Fig. 1 it will be noted that the concrete given a subsequent curing in water at 72° F. attained practically the same strength as the continuously moist-cured specimens at 28 days or 3 mo. Comparison of the curves (dotted) for air-stored specimens and the curves (dash-dot) for the specimens given warm-water curing subsequent to storage in the cold room brings out in a striking manner the necessity of furnishing moisture as well as heat. In fact, the concrete stored in water appears to be benefited more by the moisture than by the increase of temperature. For example, in the lower left-hand diagram of Fig. 1, the concrete stored in the air after 3 days exposure gained 2200 p. s. i. in the ensuing 25 days as compared to a gain of 5300 p. s. i. for the corresponding concrete stored in water at 72° F. after removal from the cold room.

The rate of gain in strength of concrete stored in water after exposure to low temperatures is approximately the same regardless of the duration of exposure. This was determined by plotting the age-strength results to a natural scale and comparing. This is not apparent from a study of the diagrams plotted to a logarithmic scale in this paper.

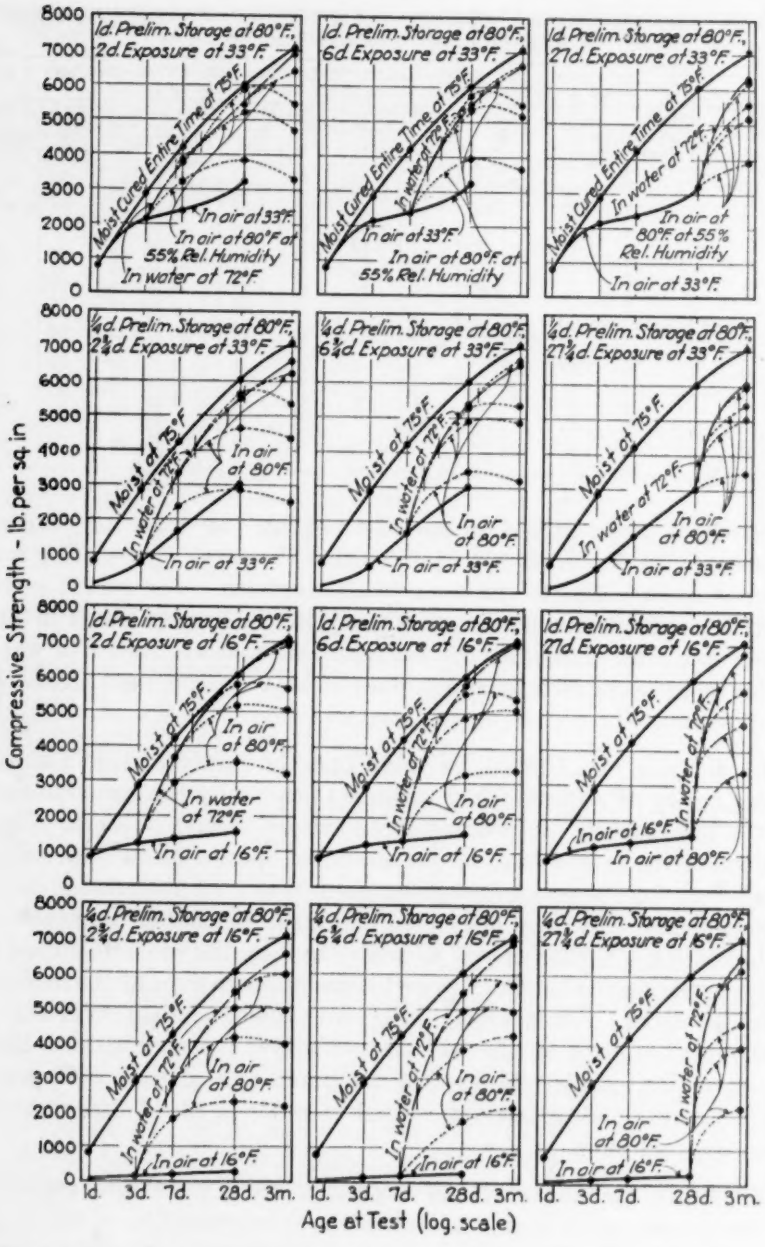
1d. Prelim. Storage at 80°F., 2d. Exposure at 33°F.
1d. Prelim. Storage at 80°F., 6d. Exposure at 33°F.
1d. Prelim. Storage at 80°F., 27d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F., 2¼d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F., 6¼d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F., 27¼d. Exposure at 33°F.
1d. Prelim. Storage at 80°F., 2d. Exposure at 16°F.
1d. Prelim. Storage at 80°F., 6d. Exposure at 16°F.
1d. Prelim. Storage at 80°F., 27d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F., 2¼d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F., 6¼d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F., 27¼d. Exposure at 16°F.
Moist Cured Entire Time at 75°F.
Moist at 75°F.
In water at 72°F.
In air at 33°F.
In air at 80°F. at 55% Rel. Humidity
In air at 80°F.
In air at 16°F.
Compressive Strength - lb. per sq. in.
8000
7000
6000
5000
4000
3000
2000
1000
0
1d. 3d. 7d. 28d. 3m.
Age at Test (log. scale)

FIG. 1—EFFECT ON STRENGTH OF WARMING CONCRETE BOTH IN WATER AND IN AIR AFTER EXPOSURE TO TEMPERATURES OF 16 AND 33° F.—LABORATORY MIXTURE. DATA FROM TABLE 1

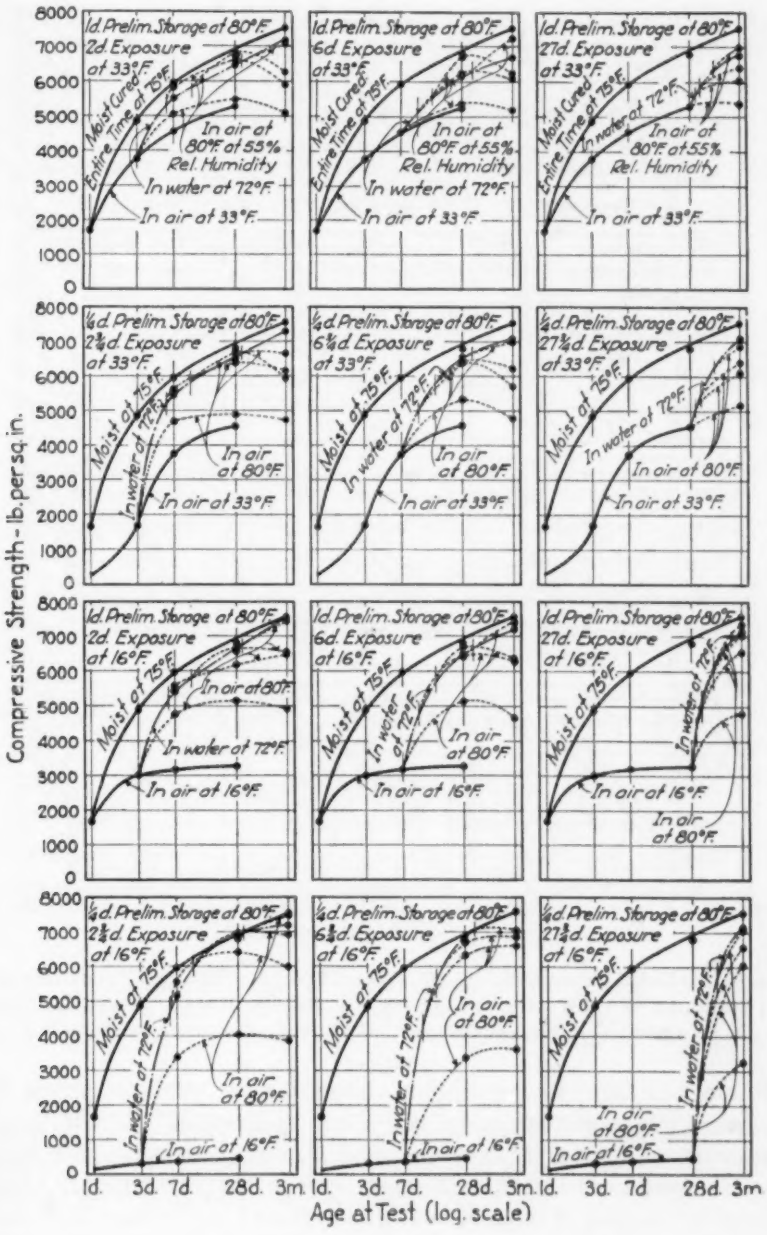
Compressive Strength - lb. per sq. in.
Age at Test (log. scale)
1d. 3d. 7d. 28d. 3m.
1d. Prelim. Storage at 80°F.
2d. Exposure at 33°F.
6d. Exposure at 33°F.
27d. Exposure at 33°F.
½d. Prelim. Storage at 80°F.
2¾d. Exposure at 33°F.
6¾d. Exposure at 33°F.
27¾d. Exposure at 33°F.
1d. Prelim. Storage at 80°F.
2d. Exposure at 16°F.
6d. Exposure at 16°F.
27d. Exposure at 16°F.
½d. Prelim. Storage at 80°F.
2¾d. Exposure at 16°F.
6¾d. Exposure at 16°F.
27¾d. Exposure at 16°F.
Moist Cured Entire Time at 75°F.
In air at 80°F. at 55% Rel. Humidity
In water at 72°F.
In air at 33°F.
Moist at 75°F.
In water at 72°F.
In air at 80°F.
In air at 16°F.

FIG. 2—EFFECT ON STRENGTH OF WARMING CONCRETE BOTH IN WATER AND IN AIR AFTER EXPOSURE TO TEMPERATURES OF 16 AND 33° F.—HIGH EARLY STRENGTH CEMENT. DATA FROM TABLE 1

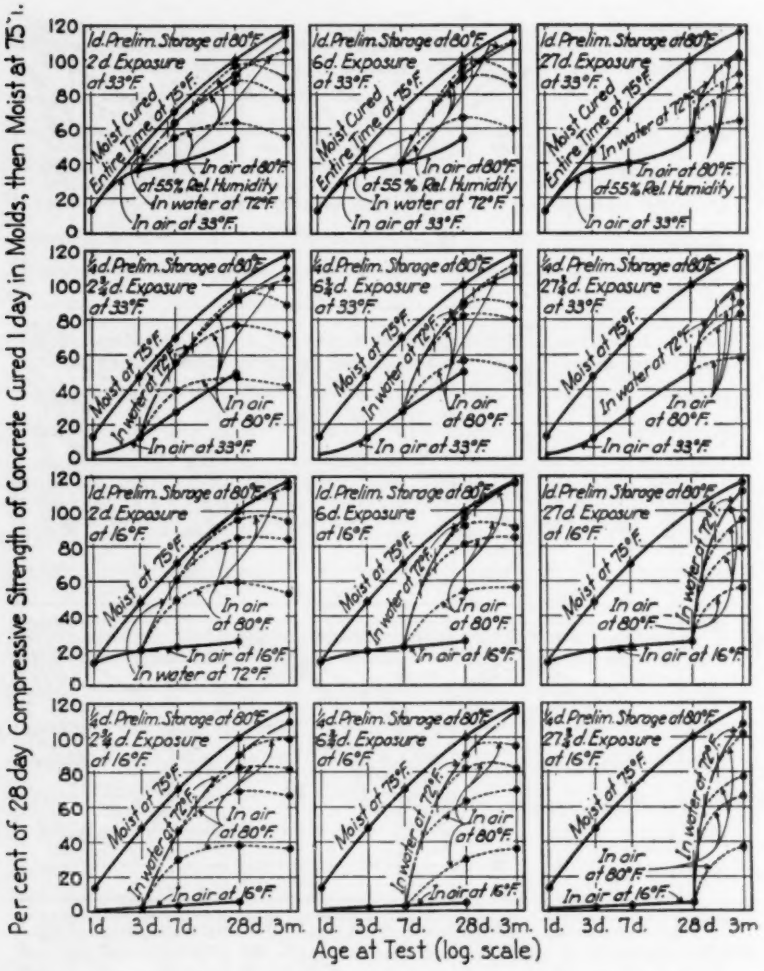
Per cent of 28 day Compressive Strength of Concrete Cured 1 day in Molds, then Moist at 75°F.
Age at Test (log. scale)
1d. Prelim. Storage at 80°F. 2d Exposure at 33°F.
1d. Prelim. Storage at 80°F. 6d. Exposure at 33°F.
1d. Prelim. Storage at 80°F. 27d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 2¾d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 6¾d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 27¾d. Exposure at 33°F.
1d. Prelim. Storage at 80°F. 2d Exposure at 16°F.
1d. Prelim. Storage at 80°F. 6d. Exposure at 16°F.
1d. Prelim. Storage at 80°F. 27d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 2¾d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 6¾d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 27¾d. Exposure at 16°F.
Moist Cured Entire Time at 75°F.
Moist at 75°F.
In water at 72°F.
In air at 80°F. at 55% Rel. Humidity
In air at 80°F.
In air at 33°F.
In air at 16°F.
1d 3d. 7d. 28d. 3m.

FIG. 3—RELATIVE STRENGTH OF CONCRETE AS INFLUENCED BY WARMING BOTH IN WATER AND IN AIR AFTER EXPOSURE TO TEMPERATURES OF 16 AND 33° F.—LABORATORY MIXTURE. DATA FROM TABLE 2

It must be kept in mind when comparing the strength curves for specimens stored continuously in the cold room at 33° F. with specimens warmed in air after exposure to 33° F. but given no water curing, that the specimens in the cold room were in the paper molds which prevented the loss in moisture such as occurred in the case of the air-warmed specimens.

Fig. 3 shows the same data as plotted in Fig. 1 except that they are expressed as percentages of the strength of the 28-day moist-cured

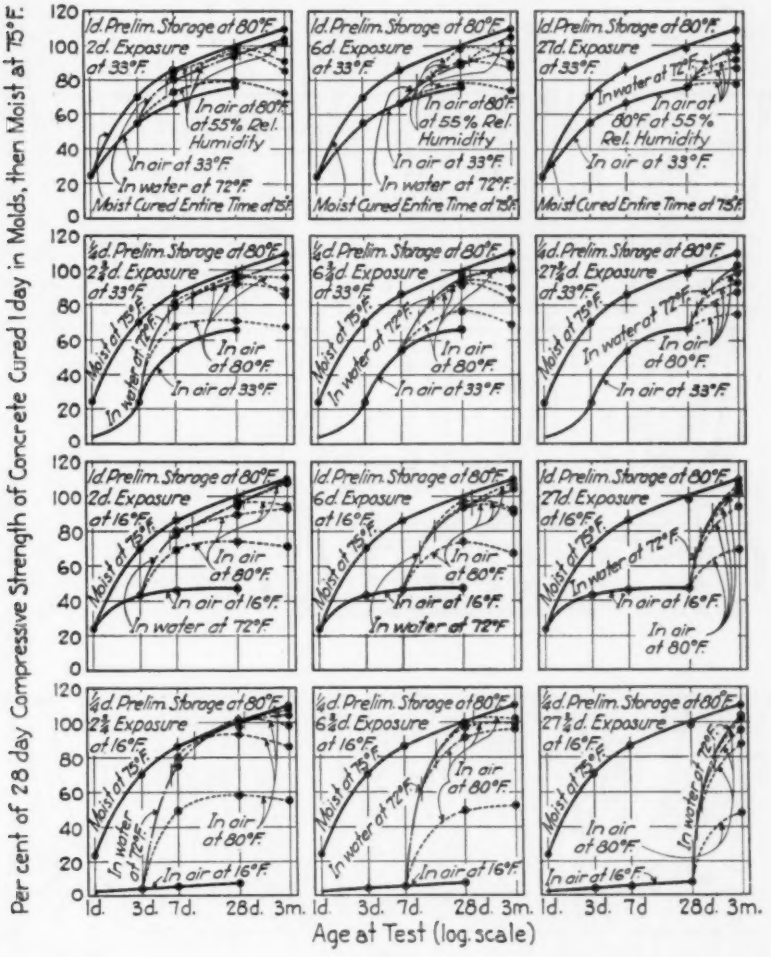
Per cent of 28 day Compressive Strength of Concrete Cured 1 day in Molds, then Moist at 75°F.
Age at Test (log. scale)
1d. 3d. 7d. 28d. 3m.
1d. Prelim. Storage at 80°F. 2d. Exposure at 33°F.
1d. Prelim. Storage at 80°F. 6d. Exposure at 33°F.
1d. Prelim. Storage at 80°F. 27d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 2¾d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 6¾d. Exposure at 33°F.
¼d. Prelim. Storage at 80°F. 27¾d. Exposure at 33°F.
1d. Prelim. Storage at 80°F. 2d. Exposure at 16°F.
1d. Prelim. Storage at 80°F. 6d. Exposure at 16°F.
1d. Prelim. Storage at 80°F. 27d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 2¾ Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 6¾d. Exposure at 16°F.
¼d. Prelim. Storage at 80°F. 27¾d. Exposure at 16°F.
In air at 80°F. at 55% Rel. Humidity
In air at 33°F.
In water at 72°F.
Moist Cured Entire Time at 75°F.
Moist at 75°F.
In air at 80°F.
In air at 16°F.

FIG. 4—RELATIVE STRENGTH OF CONCRETE AS INFLUENCED BY WARMING BOTH IN WATER AND IN AIR AFTER EXPOSURE TO TEMPERATURES OF 16 AND 33° F.—HIGH EARLY STRENGTH CEMENT. DATA FROM TABLE 2

concrete. This set of diagrams should be useful in designing concrete mixtures to determine the approximate strengths under various conditions based on any given 28-day design strength.

A noteworthy observation based on these data is the long period that the concrete can be chilled, provided no ice crystals are formed, and still develop its potential strength when given the warm-water treat-

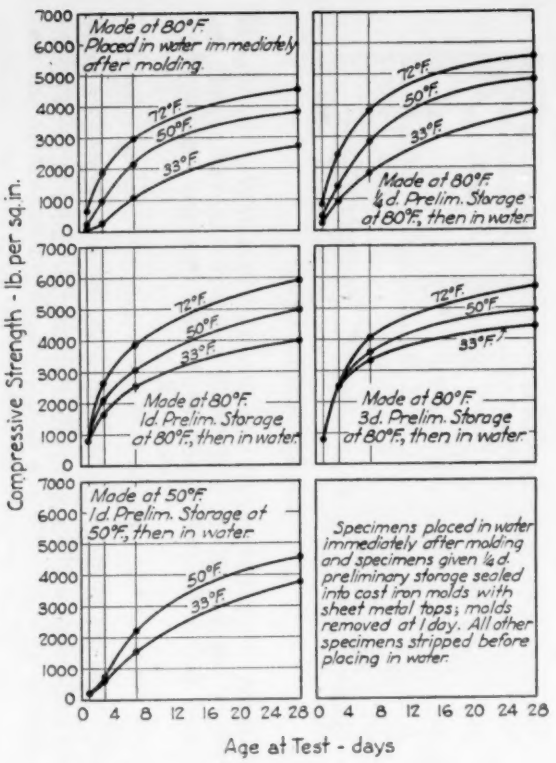
Compressive Strength - lb. per sq. in.
Made at 80°F. Placed in water immediately after molding.
72°F.
50°F.
33°F.
Made at 80°F. ¼d. Prelim. Storage at 80°F., then in water.
Made at 80°F. 1d. Prelim. Storage at 80°F., then in water.
Made at 80°F. 3d. Prelim. Storage at 80°F., then in water.
Made at 50°F. 1d. Prelim. Storage at 50°F., then in water.
Specimens placed in water immediately after molding and specimens given ¼d. preliminary storage sealed into cast iron molds with sheet metal tops; molds removed at 1 day. All other specimens stripped before placing in water.
0 4 8 12 16 20 24 28
Age at Test - days

FIG. 5—EFFECT OF TEMPERATURE OF STORAGE WATER ON STRENGTH OF CONCRETE MADE WITH LABORATORY MIXTURE. DATA FROM TABLE 3

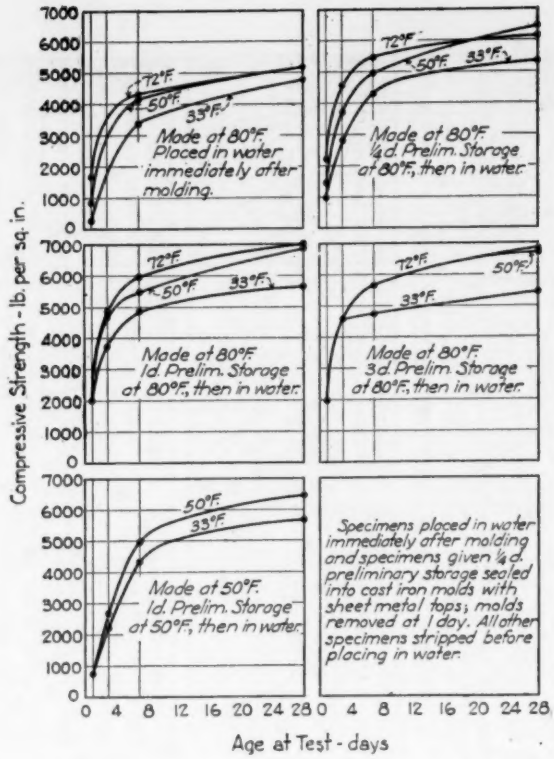
Compressive Strength - lb. per sq. in.
Made at 80°F. Placed in water immediately after molding.
72°F.
50°F.
33°F.
Made at 80°F. ¼d. Prelim. Storage at 80°F., then in water.
Made at 80°F. 1d. Prelim. Storage at 80°F., then in water.
Made at 80°F. 3d. Prelim. Storage at 80°F., then in water.
Made at 50°F. 1d. Prelim. Storage at 50°F., then in water.
Specimens placed in water immediately after molding and specimens given ¼d. preliminary storage sealed into cast iron molds with sheet metal tops; molds removed at 1 day. All other specimens stripped before placing in water.
0 4 8 12 16 20 24 28
Age at Test - days

FIG. 6—EFFECT OF TEMPERATURE OF STORAGE WATER ON STRENGTH OF CONCRETE MADE WITH HIGH EARLY STRENGTH CEMENT. DATA FROM TABLE 3

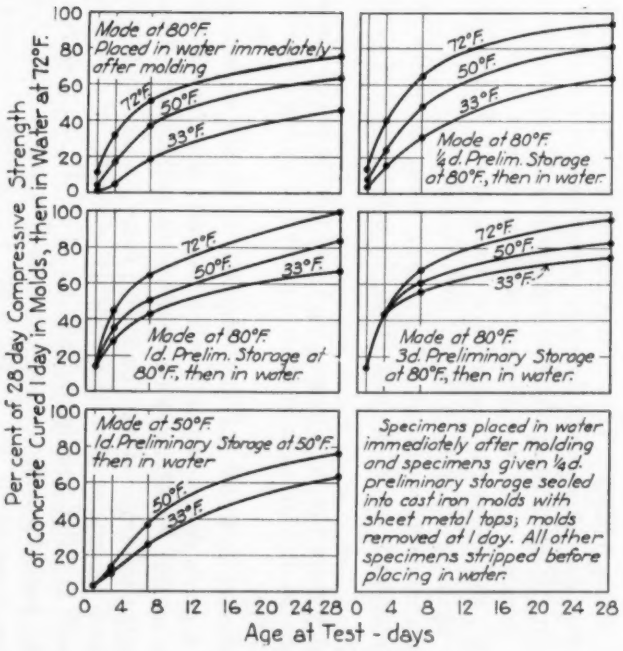
Made at 80°F. Placed in water immediately after molding
72°F.
50°F.
33°F.
Made at 80°F. ¼ d. Prelim. Storage at 80°F., then in water.
Made at 80°F. 1d. Prelim. Storage at 80°F., then in water.
Made at 80°F. 3d. Preliminary Storage at 80°F., then in water.
Made at 50°F. 1d. Preliminary Storage at 50°F. then in water
Specimens placed in water immediately after molding and specimens given ¼ d. preliminary storage sealed into cast iron molds with sheet metal tops; molds removed at 1 day. All other specimens stripped before placing in water.
Per cent of 28 day Compressive Strength of Concrete Cured 1 day in Molds, then in Water at 72°F.
Age at Test - days
0 4 8 12 16 20 24 28
0 20 40 60 80 100

FIG. 7—RELATIVE STRENGTH OF CONCRETE AS INFLUENCED BY TEMPERATURE OF STORAGE WATER—LABORATORY MIXTURE. DATA FROM TABLE 4

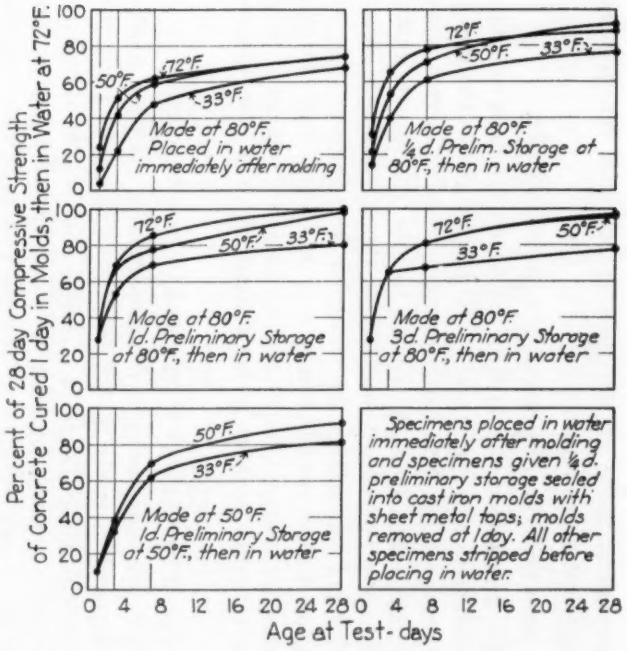
Made at 80°F. Placed in water immediately after molding
72°F.
50°F.
33°F.
Made at 80°F. ¼ d. Prelim. Storage at 80°F., then in water
Made at 80°F. 1d. Preliminary Storage at 80°F., then in water
Made at 80°F. 3d. Preliminary Storage at 80°F., then in water
Made at 50°F. 1d. Preliminary Storage at 50°F., then in water
Specimens placed in water immediately after molding and specimens given ¼ d. preliminary storage sealed into cast iron molds with sheet metal tops; molds removed at 1 day. All other specimens stripped before placing in water.
Per cent of 28 day Compressive Strength of Concrete Cured 1 day in Molds, then in Water at 72°F.
Age at Test - days
0 4 8 12 16 20 24 28
0 20 40 60 80 100

FIG. 8—RELATIVE STRENGTH OF CONCRETE AS INFLUENCED BY TEMPERATURE OF STORAGE WATER—HIGH EARLY STRENGTH CEMENT. DATA FROM TABLE 4

ment. Observations made at the time of breaking the specimens showed no imprints of ice crystals.

RESULTS FOR HIGH-EARLY STRENGTH CEMENT

Fig. 2 and 4, drawn from the data for high-early strength cement are parallel in construction to Fig. 1 and 3. As pointed out in the previous paper, the curves for the high-early strength cements have a steeper slope at the early ages than the curves for the laboratory mixture, indicating the more rapid hardening of this type of cement. Because of this more rapid hardening, the concretes at the time they were subjected to low temperatures already had developed high strengths. As was the case for normal cement, the warm-water treatment caused the concrete to attain relatively high strength in a comparatively short period.

CONCRETE EXPOSED IN WATER AT LOW TEMPERATURES

Fig. 5 and 6 give age-strength relationships for concrete made at 80° F. and exposed in water at 33, 50 and 72° F. after preliminary storage in molds for periods of 0, ¼, 1, and 3 days at a temperature of 80° F. Curves for concrete made at 50° F. and given 1 day perliminary curing at 50° F. are included. For a given storage water temperature (made at 80° F.) the lowest strengths at any given age were obtained with the concretes sealed in molds and placed in the water immediately after making. This is true for both types of cement. Under the other conditions normal cement concrete exposed at 72° F. attained about the same 28-day strength regardless of the duration of preliminary storage at 80° F. For the concretes stored in water at 50 and 33° F., the longer the preliminary curing at 80° F. the higher were the strengths at all ages up to and including 28 days.

In all cases concrete made at 50° F. gave lower strengths than similar concrete made at 80° F. and given the same period of preliminary storage.

Fig. 7 and 8 show the data from Fig. 5 and 6 in terms of the percentage of the 28-day strength of concrete cured in molds 1 day then in water at 72° F.

It is of interest to know the time required to obtain various percentages of a given design strength when concrete is stored in either air or water at 50° F. after various periods of preliminary storage. Fig. 9 is constructed from data given in this paper and in the previous paper[1]. These diagrams give the relative strength of specimens stored in water at 50° F. and in molds in air at 50° F. In the upper 4 diagrams, the basis for design is the strength of concrete cured 1 day in molds, then moist at 70° F. In the lower 4 diagrams, the basis for

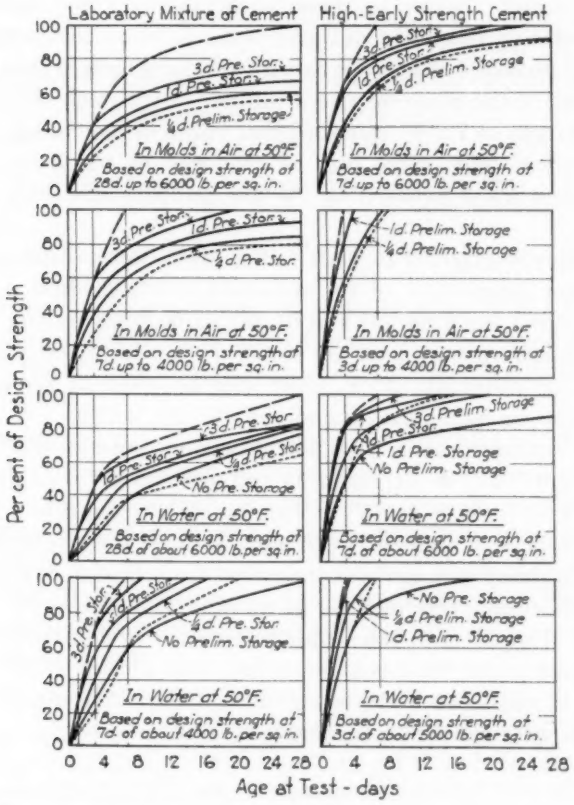
Laboratory Mixture of Cement
High-Early Strength Cement
Per cent of Design Strength
In Molds in Air at 50°F.
Based on design strength at 28d. up to 6000 lb. per sq. in.
Based on design strength at 7d up to 6000 lb. per sq. in.
Based on design strength at 7d up to 4000 lb. per sq. in.
Based on design strength at 3d up to 4000 lb. per sq. in.
In Water at 50°F.
Based on design strength at 28d of about 6000 lb. per sq. in.
Based on design strength at 7d. of about 6000 lb. per sq. in.
Based on design strength at 7d. of about 4000 lb. per sq. in.
Based on design strength at 3d. of about 5000 lb. per sq. in.
3d. Pre. Stor.
1d. Pre. Stor.
½d. Pre. Stor.
No Pre. Storage
½d. Prelim. Storage
1d. Prelim. Storage
3d. Prelim. Storage
No Prelim. Storage
No Pre. Storage
0 4 8 12 16 20 24 28
Age at Test - days

FIG. 9—STRENGTHS AT DIFFERENT AGES FOR CONCRETE STORED IN AIR OR IN WATER AT 50° F. EXPRESSED AS PERCENTAGES OF DESIGN STRENGTHS FOR WET CURING AT 70° F.

Dash lines for concrete given 1 day preliminary storage in air, then moist at 70° F. (top 4 diagrams), or in water at 72° F. (bottom 4 diagrams).

Solid lines for concrete made and given preliminary storage in air at normal temperatures, then stored in air or water at 50° F.

Dotted lines for concrete made and given 1 day preliminary storage in air at 50° F., then stored in air or water at 50° F.

design is the strength of concrete cured 1 day in molds, then in water at 72° F.

SUMMARY AND CONCLUSIONS

Principle conclusions from these tests may be summarized as follows:

(1) Where the development of normal strength of concrete has been retarded by exposures to low temperatures the concrete can be brought to practically full potential strength by saturation with warm water for a

sufficient period. While in these tests heating alone with air at 50 per cent relative humidity did give some increase in strength, it was only in the presence of an adequate supply of moisture that the potential strength of the concrete was attained. Even such short periods in water as 3 days at 72° F. were of great value in increasing strength.

(2) In these tests there was no evidence of the formation of ice crystals, even when exposed to the low temperature after 6 hours at 80° F.

(3) Concrete exposed in water at a given low temperature had a much higher proportion of its potential strength than did concrete exposed for the same period and at the same temperature in dry air.

(4) Preliminary curing in air at 80° F. for ¼, 1, or 3 days before immersion in water at 72° F. gave greater concrete strengths in all cases than when immersed immediately in the water.

(5) For concrete made with normal cement and stored in water at either 50 or 33° F., the strength at any age increased as the length of preliminary air storage at 80° F. was increased from 0 to 3 days. For concrete made with high-early strength cement, the strength increased as the preliminary storage was increased from 0 to 1 day, but 3 days preliminary storage was of little or no additional benefit.

(6) The higher strengths obtained with preliminary curing at 80° F. as compared with concrete made and placed at 50° F. for all conditions of exposure indicate the desirability of placing concrete at normal temperatures wherever possible.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for May-June 1935. Discussion should be available to the Secretary by April 1, 1935.*

Progress Report on Building Regulations for Reinforced Concrete

By Committee 501—Standard Building Code

A. W. STEPHENS, CHAIRMAN

This Committee was assigned the task of revising the "Tentative Building Regulations for Reinforced Concrete" which were tentatively adopted at the 24th Annual Convention of the American Concrete Institute, 1928.*

The personnel of the committee was very carefully selected in order that there be represented on it those best qualified to deal with the problems involved. To each of twelve sub-committees selected from the general membership, was assigned a specific section of the code for thorough study and revision. To one of these sub-committees, known as the "Editorial Committee" was assigned certain specific sections together with the responsibility for editing the work of the other sub-committees.

The work was carried on through extensive correspondence and further by means of two general committee meetings, one of two day's length held at Lehigh University in June, 1934 and the other a three-day session held in Pittsburgh, September, 1934.

The following indicates what has been accomplished with respect to each of the twelve chapters of the present tentative regulations:

Chapter 1, General: Assigned to the Editorial Committee. Report not yet completed.

Chapter 2, Materials and Tests—Article 202: Load Tests: Important changes have been suggested in the requirements for load tests and the report of the sub-committee has been approved by the General Committee. The sub-committee to which the remainder of the chapter was assigned is expected to complete its work shortly.

Chapter 3, Concrete Quality and Working Stresses—Articles 306 and 307, Allowable Unit Stresses in Concrete and Allowable Unit Stresses in Reinforcement, respectively: These have been assigned to the Editorial Committee. Report not yet completed. The sub-committee to which the remainder of the chapter was assigned is expected to complete its work shortly.

Chapter 4, Mixing and Placing Concrete: The sub-committee to which this chapter was assigned is expected to complete its work shortly.

**Proceedings*, Amer. Concrete Inst., Vol. 24, p. 786.

Chapter 5, Forms and Details of Construction: Art. 506, Concrete Protection for Reinforcement: (changed from "Protective Covering of Concrete") The sub-committee to which this article was assigned has made revisions such as to bring the requirements into accord with today's practice. Its report has been approved by the General committee. The Editorial committee to which the remainder of this chapter was assigned has not yet completed its report.

Chapter 6, Design—General Considerations: Art. 604: Resistance to Wind Forces: (changed from "Wind Loads"): The sub-committee to which this section was assigned has made its report which has been approved by the General committee. The Editorial committee to which the remainder of this chapter was assigned has not yet completed its report.

Chapter 7, Flexural Computations and Moment Coefficients: The sub-committee to which Articles 708, 709 and 710 of this chapter were assigned has devoted a great deal of study and discussion to the question of the use of definite moment coefficients in the design of flexural members, subjected to various conditions of loading and with varying conditions of restraint. Every effort is being made to formulate rules which will conform to the recent advance in knowledge concerning the action of continuous frames and still be in such form as to be readily used by the structural designer. The sub-committee to which Article 713, Floors Reinforced in Two Directions (since changed to "Slabs Supported on Four Sides"), was assigned believing the present section too conservative, has given much study to the problem of developing a section which is logical and useable. After a thorough study of several recently promulgated methods of design, the sub-committee submitted its report, based on the provisions of the proposed New York City code. This report was approved by the General committee. The Editorial committee to which other articles of this chapter have been assigned has not yet made its report.

Chapter 8, Shear and Diagonal Tension, and

Chapter 9, Bond and Anchorage

have received a great deal of study and the recommendations of the sub-committee will be submitted very shortly to the General committee for its approval. It is expected that these recommendations as approved by the General committee will be available within a few weeks.

Chapter 10, Flat Slabs—The report of the sub-committee appointed to revise this chapter has been made and approved by the General committee. No important change has been made here.

Chapter 11, Reinforced Concrete Columns and Walls: Articles 1101 to 1108, inclusive, covering the subject of columns has been thoroughly studied and it is expected that the sub-committee will submit its report within the next few weeks for the approval of the General committee. The report of this sub-committee will be of especial importance so far as the design of the spiral columns is concerned, since this design will be based upon the results of the very extensive tests that have been completed at Lehigh University and at the University of Illinois. The sub-committee assigned to Article 1109: Monolithic Walls, has completed its report which has been approved by the General committee. This report involves some important changes in connection with the design of bearing walls.

Chapter 12, Footings: The Editorial committee to which this chapter was assigned is not yet ready to report.

In addition to the sub-committees referred to above, one sub-committee has been charged with the responsibility for determining the Allowable Unit Stresses in Wire

Fabric, under Chapter 3, Article 307: Allowable Unit Stresses in Reinforcement. Laboratory tests have been made at Lehigh University and a second series of tests is now under way at the University of Delaware. It is expected that the recommendations of this sub-committee will be available by January 1, 1935.

A. W. STEPHENS, Chairman
R. R. ZIPPRODT, Secretary

Parts of the Report of Committee 501 completed by Dec. 15 for submission to the Institute membership, will be made available in mimeograph about Jan. 1, 1935. One copy of such material will be sent without charge to any member of the Institute who asks for it between January 1 and July 1, 1935. This material will be available to nonmembers (and additional copies to members) at 50 cents per copy.

Discussion of a paper by H. N. Walsh:

"Simplified Concrete Mix Design"*

AUTHOR'S CLOSURE

The author welcomes the valuable discussions by Professors Menefee and Lyse.

Professor Menefee's comparison of the author's diagrams with Fuller's curves is interesting. Fuller's work is well known in Europe, and Bolomey's grading formulas, which are much used in continental Europe, have a similar basis to Fuller's. The general agreement between the author's results and Fuller's which is shown by Professor Menefee, was to be expected, because Fuller's curves as well as Dutron's, Bolomey's, and others were used to determine the first approximations to the limiting grading curves finally adopted by the author as boundaries for his zones of gradings. The advantages that the author claims for his diagrams are that they give directly gradings of aggregates alone most suitable for use with different proportions of cement, that they render unnecessary calculations which have to be made when other formulas or curves are used, and that they can be used immediately as comparators for proposed aggregates of which sieve analyses are available.

Professor Menefee says (p. 60, Sept.-Oct. issue): "In his discussion of Table 1 the author states that the proper proportions of sand and gravel for his 1:4 mix in his Fig. 1 should be 36 lbs. of sand and 64 lbs. of gravel, but he does not say how he obtains that ratio."

The reason for taking this ratio is apparent from an inspection of the analyses of the proposed sand and gravel and the type-grading to be used, which are set out in lines A, B, and C of Table 1. The type grading shows that 38 per cent by weight of the mixed aggregate should pass a No. 4 sieve. From this one might at first conclude that the proportions should be 38 lb. sand to 62 lb. gravel, as might be the case if the gravel contained no fines. But line B shows that the gravel contains 4 per cent by weight passing a No. 4 sieve, and line A shows that 98 per cent of the sand passes this sieve. If the 38 lb. sand to

*Journal Amer. Concrete Inst., Nov.-Dec. 1933; *Proceedings* Vol. 30, p. 110. For discussion see Sept.-Oct. Journal, 1934, this volume p. 57.

64 lb. gravel were used there would be an excess of the mixed aggregate passing the No. 4 sieve. This inspection shows that the proportion of sand should be less than 38 lb.; and it is then easy to see that, in order to have 38 per cent by weight of the mixed aggregate passing a No. 4 sieve, the proportions to be taken should be about 36 lb. sand to 64 lb. gravel. The remainder of the calculation, as set out in lines D, E, and F of Table 1, shows that when these proportions are used the resulting mixed aggregate has a grading (F) very close to that of the type-curve (C). More refined calculations are unnecessary.

Professor Lyse is scarcely justified in saying that U. S. engineers have passed through the same cycle of thought as that presented by the author. European engineers also turned away from mechanical analyses (The author did so 18 years ago). Now, however, as a result of recent researches, particularly Bolomey's and Dutron's work, grading curves can be used much more effectively than was possible formerly, and European engineers have returned to the use of mechanical analyses but in ways that are a distinct advance on anything attempted in the past.

The excellence of the trial method is not to be questioned. But one saves a lot of time by being able to predict the qualities of a proposed trial mix for the aggregate grading. Recent work has made this possible. The trial mix then becomes merely a check, and a second trial is seldom necessary.

The use of the author's diagrams also helps towards economy by enabling one to decide quickly and with certainty whether two available aggregates can be mixed and used with a specified proportion of cement to produce concrete of a specified quality, and, if so, in what proportions. In some districts where aggregate industries are not developed it is difficult to get sands that are free from particles up to ¾ in., and equally difficult to get coarse aggregate free from fines. Separation into two sizes and recombination is expensive. With the help of the grading curves it is possible to determine quickly whether two such aggregates as they come from the pits can be satisfactorily combined. In many cases within the author's experience this has led to considerable economies on small jobs, and on one big job, without any sacrifice of the quality of the concrete. On large jobs where artificial grading may be desirable the advantages of having ready made grading curves are obvious. Somewhat similar sets of grading curves derived for smaller maximum particle sizes than ¾ in. and not yet published have proved very useful in helping to solve troubles in concrete products industries.

Professor Lyse has raised the question of strength formulas. Bolo-

mey's formula, as recommended by the author, is a simplification of a more elaborate formula also due to Bolomey, viz:

$$S = \left[\left(\frac{\Delta}{2.35}\right)^2 \times c/w\right]^{3/2} \times K''$$

where Δ = density (specific gravity) of the fresh concrete,
c/w = cement water-ratio by weight,
K'' = a factor depending on cement, curing, etc. and to be determined by test,
S = concrete strength (same units for K'' and S).

This gives values agreeing very closely with experimental results over a wide range of values of c/w. But the strengths of job specimens frequently vary by ± 20 per cent from the predicted strengths, no matter what formula be used. In view of this it seems unreasonable to use any but the simplest type of formula that gives close agreement with experimental results over the range of mixes ordinarily used: and the formula recommended by the author satisfies this condition.

The paper under discussion is a development of one section of an earlier and much more extensive paper read before the Inst. of Civil Engineers of Ireland in May, 1933 (Trans. Inst. C. E. Ireland, Vol. LIX, p. 275). Some of the points raised in this discussion have been treated in detail in that paper and also in articles in the issues of *Civil Engineering* (London) for November, 1933 and January, 1934.

Discussion of a paper by F. R. McMillan and T. C. Powers:

"A Method of Evaluating Admixtures"*

AUTHORS' CLOSURE

Mr. Smith's chief exception appears to be to the index of relative worth for workability as given on page 339, Table 3. In checking back over the data it was found that the actual quantities used in the six different mixes containing Celite corresponded to indexes ranging from 3.2 to about 6.5, but the ratio 5.0 held for 3 of the mixes and 6.5 for only one. Mr. Smith's higher index and other points of disagreement are partly the result of his practice of first gaging the mixes so as to have a constant flow before applying the penetration or remolding tests. Objections to this procedure were discussed in the Journal of the American Concrete Institute of September, 1932, pp. 41-48. Neither Mr. Smith nor the writers appear to have changed their opinions in regard to this question since that writing. Regarding Mr. Smith's Table 1, it is difficult to ascertain to what degree the data support his views, for while the paste volumes are given, the paste consistencies are not.

It should be recalled that there is no intimation anywhere in the paper that other investigators working with other materials and with mixes differently designed would obtain exactly the same results as did the writers. Mr. Smith's comments serve to emphasize the authors' statement on page 336 of the original paper: "From the foregoing studies it is to be expected that the results obtained in any specific series of tests will depend to a considerable degree upon the test procedure and combination of materials used."

We believe that the indexes published should not be used as a basis for specifications as suggested by Mr. Smith, at least not until just what constitutes a desirable index is more clearly established and generally understood. It might easily be inferred that because a material has the highest index for workability it is therefore most desirable. This is not necessarily true. In fact, there is much evidence that admixtures of highest value for workability are least desirable

*Journal Amer. Concrete Inst., Mar.-Apr., 1934; *Proceedings* Vol. 30, p. 325. For discussion see Sept.-Oct., Journal, 1934; this volume p. 66.

from the standpoint of other properties. Bentonite, for example, was by far the most efficient workability agent included in the series of tests but the least valuable in other respects. In general, it seemed that those materials that went farthest in making paste with water rated the highest for workability; but naturally, the resulting pastes were composed largely of chemically free water.

From this viewpoint it would seem that unless the specification writer were free to ignore all properties other than workability, specifications might eventually require that an acceptable admixture be of such nature that a stated minimum could be used without increasing the water content of the concrete. This would tend to favor admixtures of relatively low specific surface.

Discussion of a paper by R. E. Copeland and P. M. Woodworth:

"Some Tests of Load Capacity of Floors Made with Precast Concrete Joists"*

AUTHORS' CLOSURE

The authors appreciate the excellent discussions presented by Professors Dunnells and Menefee and thank these contributors for adding valuable information.

Professor Dunnells reports tests on two panels, Nos. 1-2 and 3-4, which failed with superimposed loads of 237 and 257 lb. per sq. ft. respectively. These panels were constructed with cast-in-place slabs and were similar in design to a number of panels tested by the authors where substantially greater load capacity was obtained. The difference in load capacity may be due in part to possible differences in the tensile properties of the steel used and in part to the fact that plain bars were used in the tests by Professor Dunnells whereas deformed bars were used by the authors. While the calculated bond stress with a 240 lb. per sq. ft. load is only about 210 p.s.i., any effect of the plain bar would be to reduce the bond resistance and possibly the load capacity of the panel.

Professor Dunnells also reports the results from three panels having precast slab members. These are worthy of particular attention as indicating that considerable continuity is obtained between precast slabs and joists built in this manner.

Professor Menefee contributes in Tables 1 to 5 inclusive some extremely valuable data in regard to the bond resistance of precast joists and cast-in-place and precast slabs. The 32 specimens with cast-in-place slabs reported in Tables 1 to 4 inclusive averaged 390 p.s.i. as compared with the range of 280 to 340 p.s.i. determined in the authors' tests. Professor Menefee's point that the increased strength may be due to the deeper mortice seems well taken.

The authors' conclusion that substantial metal ties as illustrated by type 6 add to the joint strength was based on observation of test specimens No. 6 in which the joint strength obtained by frictional

*Journal Amer. Concrete Inst., Mar.-Apr. 1934; *Proceedings* Vol. 30, p. 311. For discussion see Sept.-Oct. Journal, 1934, this volume p. 21.

resistance and the metal tie was greater than that of the concrete bond alone. In these specimens the concrete bond visibly failed at a stress substantially less than the ultimate strength of the joint.

Regarding the bond strength remaining after failure of the concrete bond, it is the authors' experience that there is considerable frictional resistance obtained with the morticed joint in addition to the resistance offered by the metal ties. The frictional resistance was measurable in tests of the joint strength specimens and also in the floor panels where concrete bond was prevented by a coating of paraffin-rosin solution.

Conclusion 1, Series A, purposely omits the results of specimens 3 and 6, it being the authors' opinion it would not be proper to include a value higher than that obtained with specimen 7 in reporting the range of joint strengths obtained with concrete bond only. The higher strength of specimen 6 over specimen 7 in the authors' opinion, is due to the metal tie and the same may be true to a much lesser extent in the case of specimen 3. The range indicated, 280 to 340 p.s.i. of bonding area, provides an adequate factor of safety in resisting the maximum allowable shear in the modified I-beam joist section.

Professor Menefee lists three questions which he feels should have been answered by the investigation. The authors realize the importance of these questions and feel that in large measure the answers will be found in the conclusions and data presented. Question 1 regarding continuity is covered by the following conclusion:

Conclusions Series B—1. The performance and results as to load capacity, deflection and measured stresses of panels with slab and joist connected with a concrete bond, with or without metal ties, indicated sufficient joint strength and interaction between slab and joist as to permit the use of the usual flexure formulas and allowable working stresses for T-beams in the design of floors of this type.

Question 2 regarding the necessary depth of mortice is covered, it would seem, by the test results both with the joint strength specimens and the floor panels. Question No. 3 regarding the importance of the metal tie is covered by the following conclusions of the authors' paper:

Conclusions Series A—2. Metal ties of any substantial type increase the joint strength remaining after fracture of the concrete bond.

3. Metal ties of the type used in specimen 6 may be expected to increase the ultimate joint strength above that obtained with concrete bond alone.

Conclusions Series B—7. While metal ties of the type used may be regarded as desirable supplemental connections, their use did not increase load capacity over that obtained with concrete bond alone.

The additional cost encountered by the use of metal ties will vary from practically nothing to 2 or 3 cents per lineal foot depending considerably upon the type of manufacturing equipment used.

Discussion of a paper by E. Probst:

"Plastic Flow in Plain and Reinforced Concrete Arches"*

AUTHOR'S CLOSURE (TRANSLATED BY INGE LYSE)

Prof. J. R. Shank in his discussion of my paper, which in turn was prompted by Mr. Whitney's paper,† emphasizes the necessity of a mathematical analysis of the problem of plastic flow. I agree with Professor Shank's contention that the experimental study of a problem must go hand in hand with the mathematical study; when necessary the experimental portion must come first. Regarding the problem under discussion, I have, from the valuable investigations of R. E. Davis and W. H. Glanville, and from my own experiments, reached the conclusion that the mathematical treatment of the problem cannot lead to the clarification apparently expected by Mr. Shank.

Mr. Shank replaced Glanville's formula by a logarithmic equation and seems to be satisfied that his equation is an improvement. If in Shank's last equation the actual stresses are introduced for the values of Es and n, we will observe that this formula, similar to any other formula of this nature, has no significance.

This leads me to believe that when mathematical treatments are made of problems in concrete and reinforced concrete structures, it would be well to give first consideration to the assumptions made. When the problem is studied both experimentally and mathematically, it will readily be seen that the stress condition which Mr. Shank takes as the starting point for his mathematical treatment, changes very rapidly, especially during the first months, due to the effect of plastic deformation of the concrete. For this and also for other reasons, concrete and reinforced concrete results should be obtained from experiments, not be clothed in formulas, but be made useful for the development of methods for the construction field.

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*Journal, Amer. Concrete Inst., Nov.-Dec., 1933; *Proceedings* Vol. 30, p. 137; Discussion, Sept.-Oct., 1934; this volume, p. 72.

†"Plain and Reinforced Concrete Arches," Progress Report of Committee 312, Journal Amer. Concrete Inst., Mar., 1932. *Proceedings* Vol. 28, p. 479.

Discussion of a paper by J. W. Beretta:

"Rigid Frame Concrete Bridges"*

ERRATA

Errors crept into the discussion by K. Hajnal-Kónyi, Sept.-Oct. Journal, 1934, p. 77. In the seventh line p. 78, "flat slab construction" should read: "two-way slab construction." In the last line of the discussion p. 78, "less shrinkage and greater strength" should read: "less shrinkage and greater tensile strength."—Editor

AUTHOR'S CLOSURE

The discussion from Germany by Doctor-Engineer K. Hajnal-Kónyi, gives illuminating facts on the development of long span reinforced concrete bridges in Europe and other continents. It is especially interesting and challenging to American engineers to note that long span concrete bridges have been in use in Europe since 1908, and that there are many splendid and varied examples which have followed in the ensuing years.

As stated by Dr. Hajnal-Kónyi, there is still a considerable adherence in Europe to statically determinate design systems. This is for the reason that such design systems enable adequate provision for uneven settlement of foundations, temperature changes and shrinkage in the concrete. There is no doubt that there is considerable yet to be learned regarding the application of statically indeterminate design systems to reinforced concrete. However, as this knowledge is rapidly increasing the author believes that the engineering profession will have more faith in indeterminate design systems and will be more inclined to accept in full the economies of these systems of design. It has been the experience of the author on statically indeterminate bridge designs, which are carefully studied and thoroughly supervised during erection that no trouble is experienced from uneven settlement, temperature changes or shrinkage.

As stated in the original paper the continued development of long span concrete bridges is really a problem of the chemist and physicist, rather than the engineer. If an engineer is provided with adequate

*Journal, Amer. Concrete Inst., Jan.-Feb. 1934, *Proceedings* Vol. 30, p. 196. For discussion see Sept.-Oct. Journal 1934, this volume p. 77.

construction materials, the only limits to the span of concrete bridges are those of economy and not engineering.

Dr. Hajnal-Kónyi further states it is his experience that the modern trend in long span concrete bridges is toward the use of two-way slab construction rather than girders or ribs. It is the experience of the author that the use of two-way slab construction meets very early economic limits and that the solution for longer spans is likewise dependent on the utilization of the most practical and economical details in reinforced concrete. It is almost axiomatic that the most economical and practical reinforced concrete detail capable of resisting flexure is the T-beam. In the designs prepared by the author it is this detail which has resulted in slenderness of design and reduction of dead weight. Through the utilization of the inverted T, or the soffit slab, so-called cellular construction is achieved which is considerably lighter than two-way slab construction in longer spans.

It is interesting to note in Dr. Kónyi's comments that considerable success had been achieved in railway bridge long span construction of reinforced concrete in Europe. The leader in North America in long span concrete railroad bridge construction is the Canadian National Railways, whose engineers have produced some notable advances in this type of structure.* The work done by the engineers of the new Cincinnati Terminal also merits comment as extensively advanced engineering thought in the utilization of reinforced concrete.

It is hoped American engineers will continue their research on long span concrete bridges and the next few years will see us keeping more nearly apace with our European colleagues in this type of design.

It is gratifying to the author that his paper was studied with evident interest by an experienced engineer of the U. S. S. R. It is regretted however, that the paper did not include sufficient detail to prevent some misunderstanding by the engineer, F. G. Brouyakine. It is believed that had the Russian engineer had access to the construction plans and the specifications, the design notes and data, and a record of the actual field supervision and procedure he would have changed some of his comments.

The inherent conservatism of the engineering profession in the United States of America always adds difficulties to the introduction of new design methods and construction materials. Rigid frame and continuous design is comparatively new in this country and thus has met with the difficulties of misunderstanding. In Central Europe, as indicated by Dr. Hajnal-Kónyi, such design is older and consequently

*"Tendencies in Canadian Railway Bridge Design—Recent Work on the Canadian National Railways," by H. S. Van Scoyer, JOURNAL, Amer. Concrete Inst. May-June 1934; *Proceedings* Vol. 30, p. 465.

more proven and understood. A study of the discussion of Engineer Brouyakine indicates that the development of rigid frame and continuous design is also still in a somewhat embryonic stage in the U. S. S. R. However, his frank and carefully studied discussion shows an evident interest and also shows that Russian engineers are joining those of other parts of the world in learning more of the possible advantages and economies of statically indeterminate design.

It has been proven beyond doubt that the importance of suitable foundations cannot be underestimated when statically indeterminate systems of design are used. Here again the engineer meets a challenge of accuracy and care of design for this portion of the structure. As a chain is no stronger than its weakest link so is any design, whether statically determinate or statically indeterminate, no stronger than its weakest component part. The lack of a proper analysis of any foundation which causes total or partial failure, naturally dooms the remainder of the structure to damage or failure. It is not sound economics or engineering to adhere to statically determinate systems merely because they are less susceptible to damage by poor foundations. Rather it is more important to design a foundation against all contingencies so that the economies and advantages of statically indeterminate design systems may be utilized.

Many rigid frame and continuous designs have been constructed in the United States of America and other portions of the world resting on bearings normally classed as very poor foundations. In these cases, however, great care was used in foundation analysis to insure against settlement or other damage. Most of these structures stand today as concrete proofs of the wisdom of unusual care in foundation design. In Houston, Texas, several bridges of both rigid frame and continuous girder design have been built and used successfully in sections where foundations were of the poorest and most treacherous to be found anywhere. The McKee Street bridge in particular is designed as a continuous girder with a center span of 120 ft., and all footings resting on driven piles designed for frictional resistance.

The splendid advances made by the Engineers of the Canadian National Railways in designing rigid frame bridges for railroad loadings also disproves the contentions of Engineer Brouyakine as to the limits of such spans. Here again the same principles of good rational design for both substructure and superstructure and the application of improved materials as well as proper supervision point the way to gradually increasing economic span limits. Several railway bridges in Canada having longer spans than the limits prescribed by Engineer Brouyakine are already in successful use.

Regarding temperature stresses the comments of Engineer Brouyakine again emphasizes the necessity for careful and complete design. Any design whether statically determinate or indeterminate is incomplete if temperature stresses are not fully calculated and allowed for. A neglect of this feature in any case indicates either laziness of the design engineer or a lack of full knowledge or understanding of the applied design system on his part. If all stresses whether temperature or of other kind are properly accounted for there will be no damage to the structure from these sources.

To answer the specific points brought up by Engineer Brouyakine in connection with the Martinez Street and Garden Street bridges the writer has asked W. E. Joor to make answers. Mr. Joor, as a member of the writer's organization, was engineer in charge of the design of these bridges and has the following comments to make:

Martinez Street Bridge—Mr. Beretta does not claim perfection for this structure. It is a good example of its type, though undoubtedly not perfect. However, the criticisms of our Russian friend are generally in error as for instance: The ribs do have stirrups as is very clearly shown in the diagram accompanying Mr. Beretta's paper.

There are no stirrups in the columns or piers, but their place is taken by inclined bars bent in from the main negative reinforcing where such reinforcing becomes unnecessary. These are carried down to a point in the piers beyond which the shear has become so reduced as to be no greater than is allowable on unreinforced concrete.

Referring to temperature stresses, extra steel was added all along the structure to care for these additional stresses and was found by several independent investigators who checked the design to care for these adequately.

Regarding the small radius at the inside angle between the beams and columns, it is a well known fact that in such corners the neutral axis comes down very close to the point of intersection. This was recognized and thorough study of that detail was made. A careful examination of the drawings will show that the compression of the girder is the flange of an inverted T-beam, with the flange carried back to the rear of the pier, and the pier was treated similarly each with generous compression reinforcing. These details assume the spread of the stress over a full beam space. The actual corner was rounded off with a fair sized radius that moved the extreme fiber nearly a foot away from the theoretical corner. Checking these showed all safe, but realizing the critical importance of this joint, extra precaution was taken by inserting about ten square inches of compression steel diagonally across this corner, held back by radial stirrups. All these are shown on the diagrams.

Garden Street Bridge—Mr. Brouyakine calls the end span of this structure, a cantilever. In this he is mistaken as there is always (with or without live load) a reaction on the abutment and positive moment in the span itself. The point of inflection is about 15 to 18 feet from the exterior support. The position of the point of inflection varies with the load conditions, and may shift several feet.

We agree that in event of unequal settlement of the pier and abutment, the end span would have increased stress. This is our Russian friend's main criticism. As he said earlier in his remarks, that possibility is the most dangerous feature of the

rigid frame structure. Recognizing that danger, great care was taken in the design as well as the construction of the foundation. All footings were designed for small equalized soil pressures, and the bearing soil was, for all footings, the same strata of a very hard shale. In the excavation of this shale it was necessary to use explosives. Observations have failed to show any settlement in any of the footings.

With reference to the reinforcing, this structure has no columns. The interior supports are walls extending solid the full width of the bridge. The beam stresses are distributed over the pier width by means of the soffit flange, horizontal girder and reinforcing that prevents any deflection of the pier walls between the girders. The critical corner is treated in the same way as in the Martinez Street bridge by T-beam flanges, and compression reinforcing after filleting the corner. The horizontal shear in the pier did not exceed the allowable amount for the concrete to take without reinforcing for diagonal tension. The beams do not have inclined rods. However, the shear is relatively small and amply provided for by means of vertical stirrups.

Tensile reinforcing does in some cases end in the zone of tension. Much of it extends to or beyond the point of inflection. In other cases the rods are carried several feet beyond the point where they are theoretically required and are then ended in a large radius curve. If there is any sudden increase in stress, it would not be excessively sudden and would come on bars at points where they were carrying rather low stresses already. At points where the soffit is materially changing direction due to curvature, the steel is not in tension but compression. At this point the curvature is not dangerous as such curvature is concave.

Our friend says "Greater results could be attained by using a simple statically determinate type of beam." We do not understand what is meant by "greater results." However, a simple girder design would have required larger piers and much deeper bridge construction, all of which would have materially and perhaps disastrously reduced the flow area of the stream, causing dangerous overflows. Since building, water has submerged our structure ten feet without damage. Where maximum stream flow area is required (as in the case of this bridge and the Martinez Street bridge) this type of structure is advantageous if foundation conditions permit. Bad foundation circumstances may offset the cost saving, but that cost may in some cases be neglected, if the problem be solved, to secure minimum obstructions to flood waters.

Mr. Brouyakine is comparing highway bridges against his railway practice. If we were dealing with a railway bridge our solution of the problem might be different. The relation of live to dead load has much bearing on a solution of any bridge problem, as well as the locality considered and the forces to be opposed. For instance, he refers to heavy ice floes. We do not have that problem here in the United States of America except possibly in some situations in the northern states, and even there nothing like what they have in Russia. Each bridge problem is a case unto itself, and each crossing has its own significant features. There is no one type or material that will fit all cases. Low first cost of a structure is important but not necessarily the governing factor.

As outlined by Mr. Brouyakine the Russian design, specifications, and practice are not materially different from American requirements. We certainly do not advise rigid frame or any other type of bridge construction design except by capable, experienced and careful engineers. Poor designing is bad engineering anywhere.

Discussion of a paper by A. G. Timms and N. H. Withey:

"Temperature Effects on Compressive Strength of Concrete"*

AUTHORS' CLOSURE

Bearing on Mr. Morssen's question as to whether cold weather reduces the strength of concrete permanently or temporarily, we have some later tests which indicate that the action can be considered as one of retardation only. By the application of both heat and moisture, considerable strength can be developed even during short periods. The results indicate that even in the most severe cases in our tests full potential strength of the concrete would be reached at from 3 mo. to 1 yr. by storage in water at 72° F.

Examination of concrete structures which have shown serious deterioration due to placing in cold weather has frequently disclosed imprints of ice crystals. Complete absence of such crystals from any of the specimens in the tests covered in the paper, and in the later tests for which results are now available†, indicates that some actual hardening had developed before the concrete was exposed to the low temperatures. If any considerable proportion of the potential strength of concrete is to be attained it is essential that some hardening take place before exposure.

*Journal Amer. Concrete Inst., Jan.-Feb., 1934; *Proceedings* Vol. 30, p. 159. For discussion see Sept.-Oct. Journal, 1934, this volume, p. 8.

†See "Further Tests of Temperature Effects on Compressive Strength of Concrete," this Journal issue, p. 165.

Discussion of a paper by William H. Thoman and Warren Raeder:

"Ultimate Strength and Modulus of Elasticity of High Strength Portland Cement Concrete"*

AUTHORS' CLOSURE

The data presented by Mr. Bertin upon the subject of variation of f_c' with the cement content of the paste is welcomed by the authors. His conclusion that ultimate strength might well be expressed in terms of the specific gravity of the paste seems logical. The limitations, however, mentioned by Mr. Bertin upon this method should be noted and in this connection it is well to point out that in the *RG* and *WG* mixes with c/w ratio equal to $3\frac{1}{2}$ to 1 by weight, there was less volume of water than voids in the cement.

Although the phase of the investigation discussed by Mr. Bertin was secondary to the main purpose of arriving at a relation between E and f_c', nevertheless the results of his experiments are extremely pertinent and constitute a valuable addition to the existing data in the field of concrete mixes.

Publication of Discussion Deferred

Publication of discussion, originally intended for this issue, is now scheduled for the January-February 1935 issue (this volume), under the following title:

Bonding of New to Old Concrete at Horizontal Construction Joints

*Journal, Amer. Concrete Inst., Jan.-Feb., 1934; *Proceedings* Vol. 30, p. 231. For discussion see Sept.-Oct., Journal, 1934, this volume p. 52.

Discussion of a paper by B. Moreell:

"Observations on European Practice in Concrete Design and Construction*"

CONVENTION DISCUSSION

F. E. Richart: Commander Moreell indicated that different effectiveness of spiral are permitted for different spacing; will he tell us what the effectiveness ratio would be at the spacing of one-sixth the diameter?

Commander Moreell: I figured it out for one-fifth the diameter. At one-fifth the diameter the effectiveness ratio is two and two-tenths. It is rather an involved formula and it takes some time to work it out. This is an example of the thing I pointed out—the disinclination of the French to adopt formulas for their simplicity.

Professor Richart: I tried to find out where the ratio of one-sixth came from. I think the original tests made by Considere did not give us any such specific ratio as one-sixth, but I assume it has come along as a matter of practice and has become a magic number so far as building codes and building designs are concerned. There is another rule specified for building columns—that spiral spacing shall not be more than three inches. I find instances of caissons four or five feet in diameter in which it seems desirable to use spacing greater than three inches to insure getting the concrete through the spiral.

Another thing of possible interest to the Institute members, since it was mentioned that square spirals are permitted. We have recently tested a few small columns at the University of Illinois using square and circular cross sections of the members and using the same pitch of spiral. I think the results are comparable with those mentioned here. The effectiveness of those so-called spirals and rectangular cross sections was about half as great as it was with the circular hoop which we would expect to act with uniform pressure. It does not seem reasonable that a rectangular spiral can retain its section if it is not effective, but it does produce a toughness in the column which is a thing we speak of a good deal, a feature of the column which produces

*Journal, Amer. Concrete Inst., May-June, 1934; *Proceedings* Vol. 30, p. 391.

a gradual type of failure. This factor of about one-half between the rectangular and circular types of spiral may be of interest.

Commander Moreell: The practice in France is to allow three effectiveness ratios, one for the circular spiral, one for the square band and one for the rectangular band. They have a factor of two for circular bands, a factor of one for square bands and for rectangular bands a factor b over a, where b is the smallest lateral dimension of the concrete and a is the largest lateral dimension; if you use a long narrow section, you lose a good deal of effectiveness.

A. N. Talbot: It might be well to suggest that in this rectangular spiral, the element of bond enters into it, since for the greater diameter after the spiral comes into effect as governing the action of the column, that which is farther away from the center will have a greater stress (if the size of the wire is constant), and the straightness of the side of the rectangle will decrease the effectiveness as spiral reinforcing. Also in the case of eccentric loading, these things will differ even in the circular spiral, so that the size of the wire or reinforcement and the diameter of the spiral both would enter, if you come to take account of all the elements.

S. C. Hollister: In extension of what Professor Talbot has just said: In the illustration shown, of an arch reinforced with simple tension steel members on the tension side and with the spiral on the compression side, we have an example of a curious concept of action which differs greatly from our concept in a flexural member. We have the case of a portion of the compression area of a beam marked out by itself and surrounded by the lateral reinforcement in the form of a spiral, with the idea that within this area the concrete can work safely at one stress and just outside it at a lower stress. Obviously in such a flexural member the stress in the core is varying across the cross section of the spirally reinforced portion—typical example of just what Professor Talbot spoke of, similar to the eccentrically loaded column. I was wondering whether Commander Morrell could give us any information as to any coefficient supplied for spiral use in such a manner or, in fact, for ordinary spiral columns where a large eccentricity may be applied to cause a widely varying stress over the core area?

Commander Moreell: I can not answer that question definitely. The tendency of French regulations and codes is to leave large loopholes for the ingenuity of the individual, and that particular item has not been covered in any French code I have seen. As far as I have been able to determine, the French practice is to permit an increase in the compressive working stress if the section is reinforced laterally, re-

gardless of whether the section under consideration is a column subjected to axial load only, or a column taking bending and direct stress, or the compression flange of a beam or arch. The magnitude of the increase is left to the designer but is limited by the general regulations. About the design mentioned by Professor Hollister; I am not sure that they were considering tensile stresses in the abutment under normal loading conditions; it may be that those longitudinal bars were placed as an anchorage. They frequently use a parabolic arch which conforms to the dead loading. As their live loads are lighter than ours, they usually have very little tensile stress to take care of.

Professor Hollister: Except in the case of the Freyssinet system where the crown is open and readjustment is made for shifting due to shrinkage or plastic flow, it would seem that even though the center line of the arch rib may be false according to the dead load pressure line, this region reinforced in this manner is still a critical one because of the fact that at that region there is a considerable bending moment in just the direction indicated by that scheme of reinforcement, and due in great part to the shift of the theoretical pressure line due to the lowering of the crown through shrinkage and plastic flow.

Commander Moreell: That is true, if you do not use hinges.

Professor Richart: I believe the use of these spirals in arches and other compression members is a useful thing in tying the reinforcement in place, in preventing buckling and such things as that. I have some doubt as to the usefulness of the spiral in adding to the compressive resistance of such a member. Professor W. M. Wilson, of the University of Illinois, has made a large number of arch tests, and in some of the slender arch ribs he included spiral reinforcement to see whether or not that would produce any additional resistance. As might be expected he got nothing at all from the spiral, and I think if we consider the deformation, the shortening required to bring the spiral into effective use (it is many times the deformation which exists in concrete under ordinary flexural stress) we realize that there is so much motion required there that the arch or member may fail before the spiral ever comes into play, especially if it is a long, slender member. We know that long slender columns benefit very little by spiral reinforcement because, as soon as they get into a state where the spiral works, the stiffness of the column has decreased enormously and we have a flexible member left which the spiral cannot help.

This discussion is concluded except for the author's opportunity for closure in the March-April 1935 JOURNAL.

Discussion of Report of Committee 302:

"Design of Two-Way Slabs on Beams"*

Paul W. Norton (*Boston*, by letter)—All who have interested themselves in the design of two-way construction, or have been called on to design such construction in compliance with different building codes, must have been impressed by the variety of opinion obtaining in regard to it, and have awaited with interest an authoritative pronouncement on this subject which might clarify the facts and unify practice.

Any formula proposed for the design of two-way slabs has for its object a statement of the bending moment in a unit strip of the slab in either direction, for given conditions of edge support and restraint, expressed in terms of the unit loading, the span, and the ratio between the transverse width of the panel and the span, denoted in this discussion by r.

Committee 302 presents a set of algebraic expressions for the moment coefficients, found by trial to yield results agreeing closely for specific values of the panel-ratio with results derived by theory and confirmed by test as correct. That is, the formulas are empirical, and Dr. Westergaard's derivations of the specific values constitute the rational background to which the Committee refers. The work of Dr. Westergaard, presented in his paper of 1921, and summarized by the formulas and graphs of the Committee's report, is a very valuable contribution to the study of this subject, and may be accepted as providing the closest approach to the truth about the matter that we now have.

But regarded as working formulas for practical use in designing floor construction, these which the Committee advocates are inconveniently intricate in form, and lack something of ready adaptability to problems commonly encountered, so that it is at least a debatable question whether their disadvantages are compensated by their advantages. It is disappointing that the Committee's report is accompanied by so little discussion. If the rationality it so values can be achieved only through the use of a round dozen of formulas, involving the ratio m both in numerator and in denominator, and in all powers from the

*Journal, Amer. Concrete Inst., May-June, 1934; *Proceedings* Vol. 30, p. 498.

first to the sixth, it is reasonable to suppose the merit of this rationality has been closely scrutinized by the Committee, as it will surely be by others before general acceptance.

Whether or not there may be adequate reason for preferring in working formulas to use the ratio, m, between short and long dimensions as an index of panel proportion, as does the Committee, certainly for purposes of studying the relations involved it is more enlightening to express it in terms of the ratio, r, between lateral dimension and span, as defined herein. As is brought out by Mr. Nichols in his paper presenting the proposed Boston code[1]; however obscure may be the exact rational relation between the moment in a strip and the shape of the panel, it is at least clear that M is a continuous and increasing function of r. In Fig. 1a to 12a the correspondingly numbered graphs accompanying the report of Committee 302 are redrawn to show the relation between M/wL^2 and r instead of m, the plots for the Westergaard and Boston formulas only being given. In the former discontinuities at $r = 1$ will be noted[2].

It is no part of the purpose of this discussion to defend the formulas of the proposed Boston code. The committee responsible for them has probably gone to an extreme in its zeal for simplicity; it would be hard to suggest an increasing function of r more simple than r itself, which is adopted as the factor controlling assignment of load to the strip, while r^2 enters correspondingly into the moment expression. The writer is content to let the comparative plots show how far the Boston formulas depart from the probable facts, and would be even more so if they might contribute some guidance to the choice of formulas which conform more closely to the truth and are in other respects acceptable. He is satisfied however that within proportions appropriate to two-way design the Boston formulas would yield results both safe and reasonably economical, and if rationality is to be the ground of discrimination, would be disposed to allow them some claim to preference over expressions which are not continuous functions of the ratio of panel dimensions at all.

Giving to the word rationality its literal meaning of reasonableness, there is another sense in which this quality is particularly desirable. A formula which in its form reveals the reasonable principle or conception on which it is based, may be enabled thereby to guide the judgment of the designer in adapting it to the numerous conditions

[1]Journal Amer. Concrete Inst., May-June, 1934; *Proceedings* Vol. 30, p. 504.

[2]Discontinuity in the sense of abrupt change of value is indeed avoided, but continuity in the sense here used implies that of slope and curvature as well.

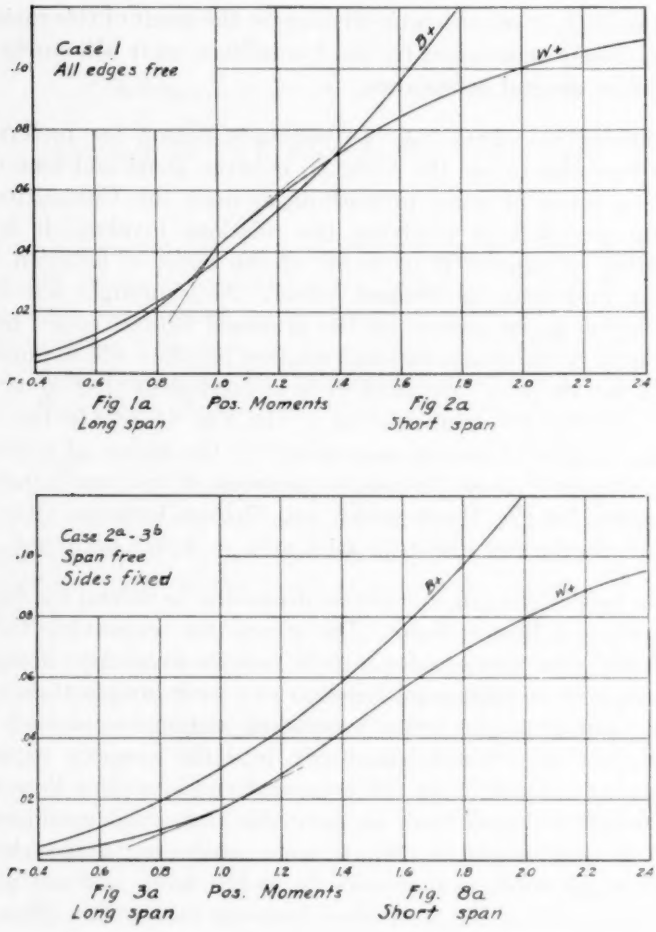
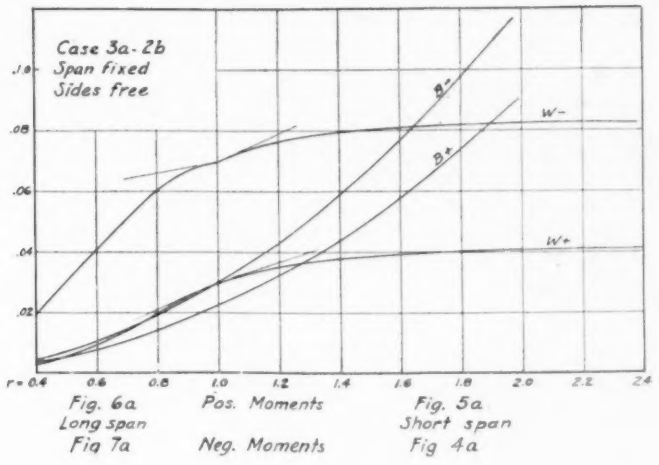


FIG. 1A, 2A, 3A, 8A

met in practice, to which it does not directly apply. For example, the formulas presented by Committee 302 tell us minutely by what algebraic processes to compute the moment in a fixed-end, long-span strip, with sides of panel both free or both fixed. It gives us no guide for the corresponding case with one side free and the other fixed, unless we can hope to arrive by interpolation at something between

$$\frac{.015m^2(1 + 3m^2)}{1 - m^4} \quad \text{and} \quad .009\ m^2(1 + 2m^2 - m^4).$$



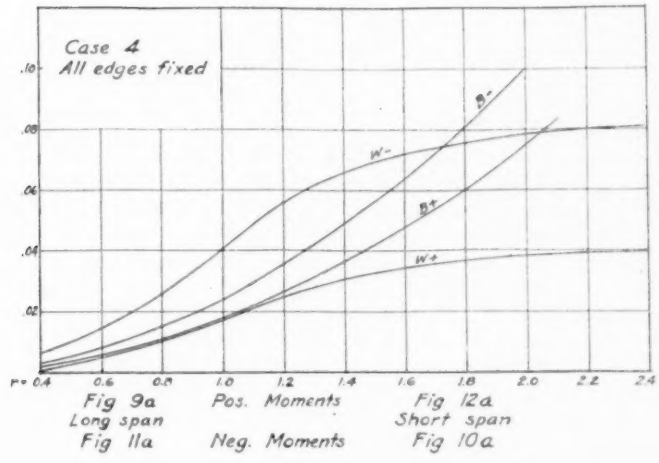


FIG. 6A, 7A, 5A, 4A, 9A, 11A, 12A, 10A

Such a specific omission as this can of course be remedied by increasing somewhat the store of formulas, and perhaps four more need not greatly increase our dismay. But there are also various degrees of restraint to be encountered between freedom and fixity. Perhaps the argument now in mind can be presented most briefly by reference to a single representative example.

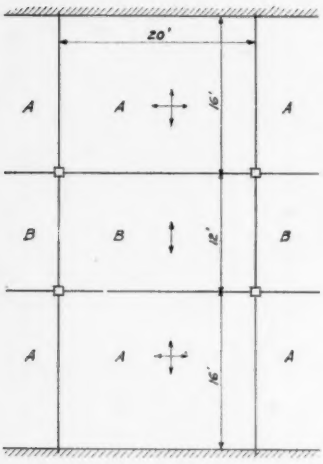


Fig. 13

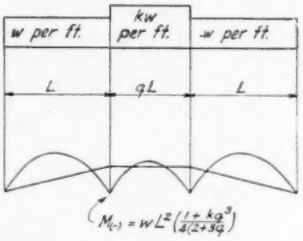


Fig. 14

FIG. 13 AND 14

Consider the moment requirements in the short-way strips of Panel A of Fig. 13, continuous at its ends with a series of similar panels, simply supported on the wall edge, and at the opposite edge continuous with the middle or corridor panel B, supposed to be of one-way construction. For simplicity of data let the dimensions indicated be the effective spans, and let the loading be 100 lb. per sq. ft. uniformly on all parts of the construction. The writer confesses himself unable to apply the recommendations of Committee 302 to this everyday problem with any confidence that he is approaching it intelligently; whereas a formula which reveals its make-up by separation of the factors involved, as does that of the proposed code of Boston, affords at least a helpful guide to the judgment. According to this code, for the short-way strip, $r = 1.25$, $C_0 = .9$, $W = \frac{1}{2}C_0wrL = 900$, or 56.25 lb. per ft. Since the adjacent panel B has a different span, and its one-way strips carry the full load of 100 lb. per ft., it is necessary to examine the special value of the coefficient C_1 for negative moment applying to the load diagram of Fig. 14. The three-moment theorem, or other analysis, leads readily to the general value of the negative moment indicated in the figure, and inserting the values $k = 100/56.25$, and $q = 12/16$, we find $C_1 = .1027$, and the negative moment is 1080 ft. lb. for a strip in the mid-half of the panel, arbitrary reduction being permitted in the outer quarters. It is hardly necessary to continue the illustration with discussion of the positive moment, and the moments in the long-way strips. The loading of the beams is derived from the elementary conception that the load carried by a strip of the slab is the load transmitted by it to its supports.

The writer believes that the characteristics of the formulas which in the absence of other available ones have made possible their application to the above example are valuable characteristics, and regrets

to find them lacking in proposals which by reason of their origin may justifiably be expected to have wide acceptance. It is to be hoped that Committee 302 will supplement its brief report by a far fuller discussion of the considerations which influenced its decisions.

E. Zbinden (*Montreal*, by letter)—In Professor Uhler's report, we have a useful compilation of formulas for the design of two -way slabs. Those formulas are based on very definite assumptions and should not be used when those conditions are not fully satisfied.

For the purpose of design, square or rectangular slabs supported on four beams are divided into two series of strips parallel to the sides. The positive bending moments are then determined for the two strips intersecting at the point of maximum deflection and the values obtained used over the width of the slab, with or without reduction for strips adjacent to supports.

Adjacent strips have not the same deflection and when the strips are rigidly connected as in a solid slab, this sets up torsional stresses in the slab and thus modifies the distribution of moments. This is sometimes called "plate action" and its best known effect is the reduction of the positive bending moments. The formulas listed in the report give the reduced moments or coefficients of reduction.

Another effect of the plate action is the introduction of negative bending moments at the corners (the elastic curve of the diagonal has its concavity on the underside near the corners). The theoretical value of those corner moments has been computed by different authors and for a square slab simply supported, the values given approximate $wl^2 : 24$. This is practically equal to the positive moment near the center of the slab.

A third consequence of the plate action is the uplift at the corners. For a square slab simply supported, its value has been given as equal to one-twelfth the total load of the slab. Special anchorage is required at the corners to prevent their uplift which would destroy plate action.

When the connection between two adjacent strips is not sufficiently rigid, plate action is impossible. In ribbed floors, for instance, the thin slab between the joists is not sufficient to resist shear caused by torsional stresses and, furthermore, such a floor cannot resist corner moments which act diagonally. The resistance to shear in a composite or combination floor is doubtful, although such a floor may be built to resist diagonal bending moments.

In those cases where plate action is impossible or doubtful, the design should be made by the method specified in Section 713 of the Joint Code which gives the moments for the case of adjacent strips independent of each other.

This discussion is concluded except for the author's opportunity for closure in the March-April 1935 JOURNAL.

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's corps of Reviewers.

Vibration as applied in placing concrete

R. F. LEGGET, *Concrete and Constructional Engineering*, Vol. XXIX, No. 6, June 1934, pp. 393-400.
Reviewed by GLENN MURPHY.

Description and discussion of vibrating equipment with special reference to Canadian practice.

Standard sand

F. RICHNER, *Tonindustrie-Zeitung*, No. 46, p. 548, June 7, 1934. Reviewed by INGE LYSE.

The author reports some compression results of mortar tests of different cements. Ordinary concrete sand mortars gave results quite different from tests with standard sand (of one grain size). It is concluded that the sand available for the concrete project is the only one to be used in the tests for the selection of the proper cement for the job.

Methods of producing artistic concrete finishes

THOMAS WALLIS, *The Structural Engineer*, Vol. XII (new series), No. 8, Aug. 1934, p. 368-375.
Reviewed by V. P. JENSEN.

Short descriptions are given of numerous methods of surface treatment. The methods are classified as, (1) those employing special materials in the concrete, or employing sliding or specially prepared forms, (2) those in which a special facing is applied to the concrete after removal of the forms, (3) those which consist of the removal of the surface film from the concrete, (4) those which include the use of precast units.

Economic design of reinforced concrete slabs

P. W. WERNER, *Teknisk Tidskrift*, No. 9, Sept. 1934, p. 101. Reviewed by INGE LYSE.

This article presents an interesting study of the economic factors involved in the design of reinforced concrete slabs. The study is based on the design stresses recommended by the 1934 Swedish Specifications with certain simplifications necessary for the solution of the problem. The economy of the slab depends upon the cost of the concrete and the reinforcement, and the working stresses in concrete and steel. Examples of design are given for different cost ratios between steel and concrete and the results are illustrated by diagrams. The author points out that the obtained results should be considered approximate and that for the given conditions the stress in the reinforcement had relatively small effect on the economy of the design.

Concrete highway construction

Issue 1934, published by German Cement Association. Reviewed by INGE LYSE.

This 96-page book describes the present German policy regarding road construction, particularly with respect to concrete roads. Clear illustrations picture the advantages of concrete highways. Descriptions of methods of construction are given in non-technical language and a statistical summary of concrete highway construction in different German states is given for the years 1925-1933. The cement-bound macadam highways are also discussed. One chapter deals with concrete highway construction in foreign countries and a summary table gives the areas constructed in a number of countries during the period 1925-1933.

Reinforced concrete and steel for large span bridges

FRANZ GLASER, *Zeitschrift des Osterreichen Ingenieur und Architeken-Vereines*, No. 30-40, 1934, p. 230. Reviewed by INGE LYSE.

The author attempts by means of approximations and assumptions to arrive at an analytical basis for the study of the economy of reinforced concrete and steel for arch bridges of large spans. He develops formulas for the economy of both materials and illustrates graphically the resulting relationships. The reinforced concrete arch bridges compete successfully for spans up to 120 m. (430 ft.) when the rise of the arch is more than 1:5 and decreases to 80 m. (270 ft.) for a rise of 1:10. While the analysis is far from perfect due to the necessary simplifications, it provides a guide for the study of the most economical material for certain types of bridges.

Watertight concrete

V. BAHRNER, *Zement*, No. 34 and 35, Aug. 23 and 30. 1934, p. 492. Reviewed by INGE LYSE.

Practical recommendations are given for the design and production of high grade concrete. The first requirement is a dense concrete, that is, a concrete with no air voids in fresh condition. This concrete may be produced by using from 15 to 20 per cent more cement paste than that necessary to fill the voids in the combined aggregates. Diagrams are presented, showing the marked effect of the type and brand of cement upon the strength of concrete, and "ideal" gradation curves for securing minimum voids in aggregates are discussed. Recommendations are also given for determining voids in aggregates. Attention is called to the effect of the maximum size of the aggregate upon the density and the effect of moisture content in sand upon "bulking."

Canteen building at Croydon gas works

Concrete and Constructional Engineering, Vol. XXIX, No. 6, June 1934, p. 377-380 Reviewed by GLENN MURPHY.

Description of the new reinforced concrete canteen building at the Purley Way works of the Croydon Gas Co. Building is 107 ft. by 79 ft. with two suspended floors, a clock tower and a basement. Design live load was 112 lb. per sq. ft. Weight is supported on 7-ft. or 8-ft. square footings on about 20 ft. centers. Basement floor was laid as a 5-in. reinforced concrete slab with a 2-in. topping. Columns are 18-in. and 20-in. square, supporting 18-in. by 11-in. girders, which in turn support 12-in. by 7-in. beams at the third points. Ground floor is 4-in. reinforced concrete slab, next floor is 2½ in. reinforced concrete slab, designed as a flange for the beams, and carrying hollow clay tiles. The roof is a 4-in. reinforced concrete slab supported by a similar system of smaller concrete beams and girders. Approximately 1410 cu. yd. of concrete and 83 tons of steel reinforcing were used.

Vibration of concrete

Staff Article, *Tonindustrie-Zeitung*, No. 68, Aug. 23, 1934, p. 816. Reviewed by INGE LYSE.

Vibration has become a very important element in the production of high grade concrete and the problem is to produce the most efficient vibration. A number of types of vibrators are already on the market and the selection of the proper one for each job is not easy. This article describes the different types of vibrators, such as form vibrators, surface vibrators, and internal vibrators. The form vibrator is driven by compressed air with a consumption of about 0.8 cubic meter per minute. It weighs about 10 kg. (22 lb.) and has 4000 vibrations per minute. The surface vibrator is a much heavier apparatus, weighing about 90 kg. (200 lb.), which is placed directly on the surface of the freshly placed concrete. This apparatus may have teeth or plugs extending into the mass during the first part of the vibration period. An internal vibrator consists of a vibration rod or spade with a handle for easy manipulation. Illustrations show the principles and make-up of the different types of vibrators.

Temperatures and internal stresses in concrete columns during setting and hardening

MALCOLM S. STANLEY, *Journal of the Institution of Engineers Australia*, Vol. 6, No. 7, July, 1934. Reviewed by J. R. SHANK.

An attempt to measure strains in column reinforcing during setting and hardening brought out that the temperature changes in the setting concrete of a 44-in. column, 1:1:2 mix, caused deformations in the steel which rendered the measurements valueless until temperature measurements on the concrete and steel produced values for proper correction. A rise of 86° F. occurred at 17 in. from the surface in 1¼ days which was not completely dissipated in 60 days. The rise at 3½ in. in was 55° in 19 hours. The temperature correction on the measured strains caused changes in sign in some cases. The corrected strain measurements on the steel of the unloaded column showed tension up to 30 days. The largest average tension occurred at 5 days and amounted to 1200 p.s.i. Measurements were made at two sides at the upper and the lower quarter points. The temperatures were taken at the mid-height. At 50 days the average was 1350 p.s.i. compression.

Fundamentals for the construction of concrete highways

W. PETRY, *Die Betonstrasse*, No. 9, Sept. 1934, p. 132. Reviewed by INGE LYSE.

A summary of concrete highway construction in Germany since 1925 reveals an immense increase from 38,000 sq. m., placed in that year to 524,000 sq. m. placed in 1933. For the year 1934 the Federal Automobile Road system alone will construct about 650,000 sq. m. Concrete highways are considered past the experimental stage and definite rules have been issued by the director of the Federal Road System. All materials and mixes must be tested before being used on any construction. Rigid construction control is established and tests made of the quality of the concrete produced. Inspection of subgrade, reinforcement and finished surface is adhered to and reported. The expansion joint is probably the most difficult problem and should be given special consideration. The joint should be at least ⅜ to ½ in. wide, filled with bituminous material. The compressive strength of the concrete must be at least 3300 p.s.i. at 7 days.

This issue of *Betonstrasse* also contains descriptions of concrete highways in Switzerland, Hungary, Czechoslovakia, Chile, U.S.A., and Sweden.

Effect of various cements on the resistance of concrete to active waters

OTTO GRAF, *Zement*, No. 27, 28, 31, 32 and 33, 1934. Reviewed by INGE LYSE.

The articles with the above title present the results of an extensive investigation of concrete made of twelve different cements, four of which were standard portland cements, one high-early-strength portland cement, three slag cements, two alumina cements and two blended cements. Chemical analysis and fineness of the cements, gradation of the aggregates and detailed description of the making, curing and testing are given. Flexure and compression tests were made in addition to exposure tests and the results are included in the report. The concrete specimens were partly submerged in sodium sulfate and magnesium sulfate waters and in sea water, and the detrimental effect measured in terms of decrease in flexural and compressive strengths. The author concludes that the durability of concrete made of portland and slag cements is more affected by the brand than by the type of cement and that the alumina cements produce less durable concrete than the portland and slag cements. The test results are presented in tables and discussed in detail.

Studies of the strength of concrete

GEORG WASTLUND, *Betong* (Journal of the Swedish Concrete Institute), No. 3, 1934, p. 135. Reviewed by INGE LYSE.

This 79-page doctor dissertation published in German with a Swedish summary presents an excellent review of the different theories of failure advanced for non-homogeneous materials such as concrete. Thorough study leads the author to conclude that the so-called "maximum strain" theory serves best for the concrete problems. When the tension strain exceeds a certain limit, the concrete will fail. This old theory of Saint Venant, which had such outstanding sponsors as Bach, and which probably can count on most adherents among American concrete engineers today, has again received a valuable support. Experimental results are in most cases in harmony with this maximum strain theory.

In addition a number of experiments were made on reinforced concrete beams having bars with hooked ends. These tests showed that a radius of curvature of about 5d is sufficient for developing the full yield-point strength of the reinforcement. The dissertation is a valuable contribution in the search for the fundamental laws of concrete and reinforced concrete.

Notes on vibrated concrete for a Dutch bridge

H. A. M. C. DIBBITS, *Engineering News-Record*, Sept. 6, 1934, Vol. 113, No. 10, p. 298. Reviewed by N. M. NEWMARK.

The article describes the use of vibration in constructing a highway bridge for a grade separation in Holland in 1933. Observations were made on displacement of reinforcement, construction of forms, deposition, consistency, density, and strength of concrete, and on general construction procedure, using outside vibrators acting on the forms.

The author recommends the use of vibrators only with caution and suggests that to prevent too many difficulties in the placement of reinforced concrete by vibration, the slump should be at least 1 to 1.5 cm. The author states further: (1) The forms require the utmost care; for the construction of walls a liberal use of form ties is recommended. (2) It is recommended that the concrete be visible during placement in order to be able carefully to regulate the time of vibrating. (3) The grooves and joints between different placements should be treated carefully or prevented altogether. (4) For important work it is necessary to have different types of vibrators at hand; external vibrators, internal vibrators, and surface vibrators.

Electric heating of concrete

Condensed from a lecture by ALBERT BRUND, *Betong* (Swedish) No. 2, 1934, p. 112.
Reviewed by INGE LYSE.

This interesting article describes the experiments and experiences gained from the use of electric heating of concrete for cold weather construction. The method seems to have passed the experimental stage and achieved official recognition, particularly in Soviet Russia. The results are so encouraging, that the concrete interests cannot neglect giving it the most serious consideration. The method consists of heating the freshly placed concrete by means of electric resistance. Alternating current of low voltage (about 20 volts) is used and the electrodes consist usually of sheet metal strips about 1/50 in. thick placed on the exposed surface of the concrete. The amount of current required for heating one cubic yard concrete one degree Farenheit may be assumed to be approximately 0.5 kwh. The economy of this type of heating the concrete during cold weather construction exceeds all other methods under Russian conditions, the cost varying from 2½ to 12 per cent of the cost of the concrete. Very successful results were obtained by electric heating on a job where the temperature ranged from 14° F. to —13° F. The heated concrete attained about 75 per cent of its 28 day strength at the age of 36 hours. The strength of the concrete was therefore much greater than for ordinary cured concrete at early ages and at 28 days it was still about 20 per cent above the ordinary concrete. Details are given of method of arranging electrodes as well as of cost of heating.

Floating mixing plants will concrete Wheeler dam

LEE H. HUNTLEY, *Engineering News-Record*, Oct. 4, 1934, Vol. 113, No. 14, p. 431.
Reviewed by N. M. NEWMARK.

The Wheeler Dam, which the Tennessee Valley Authority is constructing across the Tennessee river at the head of Lake Wilson, will be built upon a site where the river is 6400 ft. wide. The abutments are vertical cliffs rising directly from the water about 100 feet as compared with an overall height for the dam of 70 ft. To span the river with a cableway for placing the concrete would entail excessive expense. Accordingly, since Lake Wilson is navigable and also during part of the year navigation is available from the mouth of the Tennessee on the Ohio river to Lake Wilson, floating mixing plants and barge delivery of concrete materials will be employed in building the dam.

Concrete mixing plants will be placed on barges and the barges moored to the cofferdam at desired points. Cement and concrete aggregates can be brought up the river by barges and dumped directly into the mixing plants by cranes mounted on the mixer barges. During the high-water seasons the aggregates will be barged directly from the dredge to the dam site. To provide a source of supply during the low-water seasons, the contractor has arranged to store considerable quantities of sand and gravel at a point a few miles below the Wilson Dam where sufficient channel is available at all times for transportation to the dam site.

Electrical heating of concrete

C. KUNZ, *Le Ciment*, Aug., 1934. Reviewed by P. H. BATES

This article is a reprint of one which appeared in *Bulletin Technique de la Suisse Romande*, Mar., 1934. The author presents data and results obtained in an unidentified laboratory in Switzerland and in some also unidentified jobs in that country by electrically heating concrete in cold weather to prevent freezing. The work apparently was instigated as a result of work done in Sweden using electric current for this purpose.

The author says that by transforming the usual line currents down to voltages

of from 20 to 50 by the use of suitable transformers, it is possible to heat newly placed concrete far more economically and advantageously than through the methods at present used of pre-heating aggregate and of subjecting the freshly made concrete to steam heating. As a result of some observations wherein the surrounding temperature varied from —8 to +20° C. the author calculates the consumption of current would amount to 1 kwh per degree per cubic meter of concrete.

The current is led from the transformer to electrodes which may be plates placed upon opposite sides of the concrete as in piles or plates or grills placed on the opposite sides of retaining walls, or very thin plates inserted at various places in the concrete and allowed to remain in the concrete after setting.

Some results are presented showing the temperature which had been attained and the current consumption over a period during which some fresh concrete had been submitted to the action of the current. It is, of course, essential to use an alternating current to avoid electrolysis of the water.

Cooling Boulder dam concrete

BYRAM W. STEELE, *Engineering News-Record*, Oct. 11, 1934, Vol. 113, No. 15, p. 451.
Reviewed by N. M. NEWMARK

The system of artificially cooling the concrete and of grouting construction joints in Boulder dam is described in this article. Under ordinary conditions of concreting, a large element of volume change results from the expansion of the concrete due to the heating during the hardening process and the subsequent shrinkage as the concrete slowly cools. In a very large mass this cooling occupies an extended period, and the concrete continues to shrink and tends to crack in various unexpected and undesired ways.

By artificial cooling this shrinkage is concentrated in a relatively short period. The provision for definite radial and circumferential shrinkage joints within the concrete localizes the change of shape and prevents cracking elsewhere. When these joints are filled with grout, after the cooling is completed, the concrete is converted into a monolithic mass at a time when it is in the state corresponding to the permanent temperature of the dam. The cooling of the concrete in the dam is accomplished by circulating air-cooled and refrigerated water through pipes buried in the concrete.

Grouting of the contraction joints is accomplished by forcing a neat cement grout through a system of pipes with outlets in the contraction joints, at intervals of 30 to 50 sq. ft. Various grout pressures are used up to 300 p.s.i., but a pressure of 100 p.s.i. has been demonstrated to be sufficient to drive the surplus water into the concrete on either side of the joint and give a grout film of satisfactory density.

Cements containing blast furnace slag

P. DUMESNIL, *Revue des Materiaux de Construction et de Travaux Publics*, May, June, July and Aug. 1934.
Reviewed by P. H. BATES

This series of articles on slag cements should be of much value to those in this country where interest has been aroused recently through the appearance on the market and the use of pozzuolanic-portland cement mixtures, such as are being used in part on the Golden Gate bridge and the San Francisco-Oakland bridge at San Francisco.

The author divides the series of articles into ten chapters, the headings of which will furnish a very excellent means of appreciating what he presents: "Blast furnace slag"; "Theories regarding the nature of blast furnace slags"; "Origin of metallurgic cements" (the term metallurgic cement is used in France to include cements made

wholly or partically from blast furnace slag); "Classification of slag cements"; "Manufacture of metallurgic cements"; "Properties of slag cements"; "Their use"; "A discussion of certain particular conditions under which these cements should be used"; "Nature of, and plant for producing these cements"; and "Development of Standards."

The titles of these various chapters will indicate possibly that the articles are intended more for those interested in the production of such cements than those interested in their use. However, though this type of cement is practically unobtainable in the United States, the articles are of interest, since other artificial pozzuolanic materials, such as calcined clay or shale, natural cements, and hydraulic limes are attracting the attention of many and such a pozzuolanic material as quenched blast furnace slag will undoubtedly appear on the market in this country just as it has abroad.

Boulder dam cement and concrete studies

R. F. Blanks, *Engineering News-Record*, Nov. 22, 1934, Vol. 113, No. 21, p. 648.
Reviewed by N. M. Newmark

This article summarizes briefly some of the outstanding information of general interest concerning the intensive research program on cement and concrete that has been an important factor in the Boulder dam project. The complete report on this program will require nine printed Government bulletins that are now in preparation. The present report contains 17 sets of curves presenting data obtained on properties of low heat cement, specific heat and conductivity, compressive strength relation to aggregate and cylinder size, cement efficiency (unit strength per unit of cement content), the effect of wet screening, and the effect of placing temperatures.

The low heat cement for Boulder dam is practically identical in physical properties to standard portland cement. However, a wide range is represented in the potential compound compositions of the two types. The low heat cements are characterized by high percentages of di-calcium silicate and tetracalcium alumino-ferrite with relatively low percentages of tri-calcium silicate and tri-calcium aluminate as compared with relatively high values of the latter two for standard cements.

The effect of size of test specimens on compressive strength and other properties of concrete has been evaluated for cylinders ranging from 2 x 4 in. to 36 x 72 in. The relationship between maximum size of aggregate and various properties of concrete has been established for mixes in which the maximum size of aggregate varies from ¾ in. to 9 in.

Much needed information pertaining to the characteristics of mass concrete has been obtained in this investigation. All the subjects touched upon in this brief report and many other studies will be treated in detail in the proposed series of government bulletins.

New Swedish government specifications for cement and concrete of 1934

Cement och Betong, No. 4, Oct. 1934, p. 21. Reviewed by Inge Lyse

The new Swedish specifications are divided into two major sections, one on cement and one on concrete. Among the new requirements worth noting is one specifying that the cement sacks must be numbered or otherwise identified so that control of date of sacking may readily be made. The quality of the cement is determined by fineness test, time of set, volumetric observation and strength test. Less than two per cent of the cement shall be retained on a 900 mesh sieve (mesh per cm^2) and less

than 15 per cent on the 4900 mesh sieve. Both compressive and tensile strengths are required. The portland cements are divided into three groups: group A with strength of 200 p.s.i. at 7 days moist curing; group B with 3900 p.s.i., and group "special" with 3550 p.s.i. at 3 days and 5300 p.s.i. at 7 days moist curing. The alumina cement must have a strength of 6400 p.s.i. at 24 hours moist curing.

The concrete section requires that the sand shall be tested for organic matters because nine out of ten concrete disintegrations have been found due to organic matter. The Abrams-Harder test is recommended for testing the sand. The gradation of the aggregates is permitted to vary within rather broad limits and the quality of the concrete is expressed by cement content per cubic yard of concrete and strength at 28 days.

For reinforced concrete construction the permitted maximum compressive stresses in flexure must not exceed 1/3.5 the 28-day compressive strength, while for plain concrete the compressive stresses must not exceed 1/5 of the compressive strength of the concrete. For direct compression the permitted stresses must not exceed 80 per cent of the permitted flexural stresses.

Warping of concrete pavements

W. V. BUCK, (State Highway Engineer, Kansas State Highway Department), *American Highways*, July, 1934, p. 9. Reviewed by HIGHWAY RESEARCH BOARD

This is a report on the progress of a continuing investigation of the warping of concrete pavement slabs. The warping was first observed on several projects in 1928 and has been under investigation since that time both in the field and the laboratory. Holes have been drilled through the slabs on a line four feet from the pavement centerline and at distances of 1, 5, 15, 30 and 50 feet from transverse joints. The samples of subgrade taken through the holes were tested for density and moisture content, along with other physical and chemical tests. Precise levels have shown the actual movements of the slabs.

Classification of soils under warped slabs show them to be 7 soils which might be called "border line" soils. They are clay soils with reasonably high shrinkage and usually contain 30 per cent or more of grain sizes below .005 mm. and 60 per cent or more smaller than .05 mm.

Indications from the investigation so far are that water enters the joint causing increase in volume of the soil. As the distance from the joint increases, the moisture content decreases. The greatest warping occurs at the point of maximum subgrade moisture. In some cases where no edge curb was used, water penetrated the soil as much as four feet from the edge of the pavement with consequent volume change sufficient to cause entire loss of crown and resulting in longitudinal cracking near the line of moisture change in the soil. Where edge curb allows little water to enter along the edge of the slab, no such warping has occurred. A detailed report will be made by the Kansas State Highway Department when the investigation has been completed. (A special subcommittee of the Maintenance Committee of the Highway Research Board will make a report on warping of concrete pavements at the December meeting of the Board).

Tests of models of arch dams and auxiliary concrete tests

J. L. SAVAGE, I. E. HOUK, H. J. GILKEY and FREDRIK VOGT, Vol. II, *Engineering Foundation Arch Dam Investigation.* Reviewed by H. J. GILKEY

Reports results of tests conducted at the University of Colorado 1927 to 1929 by the U. S. Bureau of Reclamation on concrete models of the Stevenson Creek and Gibson dams preparatory to model tests on Boulder and other dams which have been

in progress since that time. The Stevenson Creek dam was of very slender section, the resistance being developed almost entirely by horizontal arch action. The Gibson dam and model are of the heavy arch or arch gravity type. The behavior of the first model was compared with test results obtained from its heavily loaded prototype, the Stevenson Creek test dam in California. The results from the second model were checked against the predicted behavior as computed by the trial load method of analysis. In both instances, the results obtained were remarkable in their agreement with calculated expectations. Beyond this general statement, it is impracticable to detail the findings or methods of test procedure, other than to add that the latter were highly original and unique in many respects.

A second and more or less independent phase of the investigation was the extensive auxiliary program of tests of the concrete mixtures from which the models were made. Stresses at different points in a dam may be compressive, tensile, and shearing. Time loading and volume changes are important in addition to the elastic and non-elastic physical properties. The modulus of elasticity in tension, flexure, compression and shear is important as is Poisson's ratio and the effect of repeated loadings. It was expected originally that the models would be tested air-dry but it proved desirable to keep them saturated throughout the tests. In conformity with the original plan the auxiliary tests were made to include a variety of different curing combinations and conditions of test. For several different ages, curings, and test conditions, stress strain relationships and both the elastic and non-elastic properties were determined in compression, tension, flexure, and torsion. Volumetric changes were observed on 40 in. bars and deformations and deflections due to sustained loads were determined. Some repeated partial and ultimate tests were also reported. Again it is possible here to indicate only the scope rather than the results of the investigation.

Chemical reactions of seasoning, reversion and restoration

PAUL S. ROLLER, Nonmetallic Minerals Experiment Station, U. S. Bureau of Mines, New Brunswick, N. J., *Industrial and Engineering Chemistry*, Vol. 26, p. 1077-1083, Oct. 1934.

Reviewed by ROY N. YOUNG

Six different clinkers were used in a study of the relation between setting time and exposure of the clinkers to water vapor and carbon dioxide. The chemical reactions of seasoning are expressed by equations (2) and (4) given above and equation (7).

$$4CaO.Al_2O_3.XH_2O\ (s) + 3CaSO_4 \rightarrow 3CaO.Al_2O_3.3CaSO_4.XH_2O\ (s) + Ca(OH)_2' \quad \ldots\ldots (7)$$

All of the clinkers were affected by the various exposures in the same way qualitatively and in some respects quantitatively. With increased seasoning or absorption of water vapor in excess of carbon dioxide they changed from slow to quick set. With an apparent increased formation of reactive calcium hydroxide in the liquid phase (reactive calcium hydroxide refers to the excess calcium hydroxide in solution above the concentration at which tetracalcium aluminate becomes the stable phase.) This increased reactive calcium hydroxide, through seasoning, is explained by activation of tricalcium silicate by absorbed water vapor, which results in more rapid hydrolisis of the tricalcium silicate to furnish more rapidly calcium hydroxide. Thus the tricalcium silicate becomes more effective as a retarder. Clinkers exposed to carbon dioxide in excess of water vapor become quicker setting. Carbon dioxide reacts with the potential calcium hydroxide in the tricalcium silicate—water vapor complex, resulting in deactivation of the tricalcium silicate with respect to its hydrolisis and, hence, in acceleration of the setting.

The effectiveness of carbon dioxide in causing reversion of set was dependent upon

the state of seasoning. Also under given conditions the effectiveness of carbon dioxide varied among the several clinkers indicating that some clinkers have an inherent susceptibility to quick setting with exposure to carbon dioxide. It appeared that a relation existed between this phenomenon and chemical composition.

Reversion from slow to quick set was obtained by heating at sufficiently high temperature, which in accordance with the author's theory should deactivate the tricalcium silicate. Restoration of slow setting was affected by exposing to humidified air, clinkers that had become quick setting because of the action of heat or carbon dioxide.

Experiments with concrete in torsion

PAUL ANDERSON, *Proc. Am. Soc. C. E.*, Vol. 60, No. 5, May 1934, p. 641. Reviewed by H. J. GILKEY

Examples of concrete members subjected to torsional stress are longitudinal balcony girders, exterior floor beams, skew arch under vertical load, etc. Reference is made to a few previous tests of plain and reinforced concrete in torsion by European and Canadian investigators to which might now be added recently published torsional tests of plain concrete performed in 1928 and 1929 and reported by the Engineering Foundation in Volume II of the Arch Dam Investigation (pp. 451-465 and 500-508). Mr. Anderson tested a total of 48 specimens of which there were: 6 circular of plain concrete; 9 of square cross-section reinforced with ½ in. square corner rods only; 9 as above with addition of $\frac{5}{32}$ in. round ties spaced at 6 in.; 9 as above except ties spaced at 3 in.; 6 with 45 deg. spirals spaced 3 in. normal to 45 deg. direction; 9 as above except spirals spaced 2 in.

Hard grade steel (E.L. 79000 p.s.i.) was used for the corner rods and soft annealed black wire (E. L. 43,000 p.s.i.) was used for ties and spirals.

Compr. Str. (Ult.) (p.s.i.)	Torsional Str. (Ult.) (p.s.i.)	Mod. Elast. in Shear (p.s.i.)
2125	275	1,400,000
3300	330	1,500,000
5200	420	2,140,000

The strongest of the reinforced specimens developed about three and two times the resistance of plain concrete for the 2,000 and 5,000 p.s.i. mixtures respectively. For each type of reinforcement the resistance increased with the strength of the concrete.

The author concludes that:

1. Modulus of elasticity in shear varies with the strength of the concrete.
2. Concrete in torsion fails in tension and plain concrete should not be subjected to a torsional stress in excess of half its ultimate tensile strength (factor of safety of 2).
3. All of the devices tried increased torsional resistance with their effectiveness increasing in the order of their listing. The 45 deg. spirals were most effective and the corner bars without ties are not recommended.

Hydration of portland cement compounds

R. H. BOGUE and WILLIAM LERCH, Portland Cement Association Fellowship, National Bureau of Standards, Washington, D. C. *Industrial and Engineering Chemistry*, Vol. 26, No. 8, Aug. 1934, p. 837-847. Reviewed by ROY N. YOUNG.

This paper presents thoroughly the procedures and results of an extensive investigation of reactions taking place when the compounds of cement, individually and collectively, are gauged with water in proportions similar to those used with cements

in concrete. Studies have been made of the chemical nature of the reactions of hydrolysis and hydration, the rate of reactions, the nature of the reaction products, the physical structure of the hardened specimens and the compressvie strengths and other physical characteristics of the materials. Also a general theory has been developed which permits an understanding of the chemical and physical mechanism of the hardening process. The effects of the aluminates on the silicates is indicated both in the absence and in the presence of a retarder. The following is a summary of the conclusions:

The tricalcium silicate and beta dicalcium silicate are chiefly responsible for high compressive strength in portland cement. Tricalcium silicate reacts rapidly with water under the conditions of the tests,forming crystalline calcium hydroxide and amorphous hydrated calcium silicate of composition approaching $2CaO.SiO_2.XH_2O$. The beta dicalcium silicate hydrates to form the same amorphous hydrated silicate, but only traces of crystalline calcium hydroxide have been observed. The rate of development of compressive strength appears not to be governed by the total fixed water of hydration, but largely by the structure and nature of hydration of the colloidal calcium silicate hydrate. A distribution of water is an essential factor as well as the total amount of water in combination. The compounds containing alumina react rapidly with water to form fluffy crystalline hydrated calcium aluminate of the composition $3CaO.Al_2O_3.6H_2O$.

In the absence of a retarder the alumina containing compounds tend to raise the early strengths and lower the late strength of the mixtures, which is explained by the resulting tendency to rapid development of more or less weak and open structure formed by the crystalline tricalcium aluminate hydrate. The $4CaO.Al_2O_3.Fe_2O_3$ reacts with water to form an amorphous hydrate and crystalline $3CaO.Al_2O_3.6H_2O$. Gypsum reacts with alumina with the formation of crystalline calcium sulfoaluminate, which prevents the formation of the hydrated tricalcium aluminate and thus retards initial set. Gypsum tends to counteract the influence of alumina in lowering the compressive strengths of the calcium silicates.

Portland cement pastes

R. H. Bogue, William Lerch, and W. C. Taylor, Portland Cement Association Fellowship, National Bureau of Standards, Washington, D. C., *Industrial and Engineering Chemistry*, Vol. 26, No. 10, Oct., 1934, p. 1049-1060. Reviewed by Roy N. Young.

This paper presents in detail results obtained in extensive investigations which indicate the effects of cement composition on the volume changes of pastes stored in water and air and on their resistance to the action of solutions containing salts which may be encountered in natural waters. Volume changes are considered from the viewpoint of hydration, colloidal swelling, and the formation of secondary products. Salt action is considered from the viewpoint of the formation of additional products and reactions of base exchange. The conclusions reached from these studies are as follows:

1. Expansion of cement pastes stored in water is a function of the content of $3CaO.Al_2O_3$ (or ratio of alumina to ferric oxide), sulfur trioxide, magnesia, and free calcium oxide. The effect of the $3CaO.Al_2O_3$ content has been found to be slight when present in computed amounts of 7 per cent or less. (In the cements used, this corresponds to a ratio of alumina to ferric oxide of about 1:3.) The influence of sulfur trioxide is slight when present in amounts not exceeding 2 per cent, and larger quantities are ineffective provided the $3CaO.Al_2O_3$ content is low. The magnesia may produce excessive expansions at ages beyond 3 months when present in amounts

above about 5 per cent. The free calcium oxide may result in excessive expansions during the first week when present in amounts above about 2 per cent.

2. Contraction of cement pastes stored in air increases with increasing $3CaO.Al_2O_3$ content (or ratio of alumina to ferric oxide).

3. Susceptibility to attack by solutions of sodium and magnesium sulfate is found to be determined primarily by the $3CaO.Al_2O_3$ content (or ratio of alumina to ferric oxide) of the cement. When the $3CaO.Al_2O_3$ is present in amounts computed to be 7 per cent or less (ratio of alumina to ferric oxide of about 1:3 in the cements studied), the specimens show high resistance to sulfate solutions, but the resistance diminishes rapidly as the $3CaO.Al_2O_3$ content (or ratio of alumina to ferric oxide) increases above the amounts given previously.

The presence of magnesia in amounts above about 5 per cent increases slightly the susceptibility to attack by sulfate solutions.

4. Changes in the $3CaO.SiO_2$ and $2CaO.SiO_2$ contents of the cement have been found not to alter the order of magnitude of the volume changes or resistance to sulfate solutions except that very low $3CaO.SiO_2$ cements of high $3CaO.Al_2O_3$ content may be attacked more rapidly by sulfate sulutions than cements of higher $3CaO.SiO_2$ content containing the same percentage of $3CaO.Al_2O_3$.

5. Specimens of the cements prepared of lean mix and high water-to-cement ratio were found to be less resistant to the action of sulfate solutions than those made of a rich mix and low water-to-cement ratio. However, certain changes in cement composition were found to have a greater effect upon resistance to sulfate attack than changes in the richness of the mix and water-to-cement ratio, within the limits studied.

Effect of cement composition on mortars and concretes

H. F. GONNERMAN, *Engineering News-Record*, Nov. 22, 1934, Vol 113, No. 21, p. 651.
Reviewed by N. M. NEWMARK.

This paper is a report of tests made in the Research Laboratory of the Portland Cement Association, Chicago, using 74 laboratory prepared portland cements covering 72 different compositions. The investigation represents a comprehensive study of cement composition in relation to its effect on strength, length changes, resistance to sulphate water and to freezing and thawing of mortars and concretes. The author's conclusions are as follows:

1. Strength:

(a) The principal strength-giving constituents of portland cement are the tricalcium and dicalcium silicates. Tricalcium silicate (C_3S) not only contributed much of the strength at the early ages but continued to be effective in increasing strength at the later ages also. Dicalcium silicate (C_2S) contributed little to strength up to about 28 days but was largely responsible for the increase beyond that period.

(b) Tricalcium aluminate (C_3A) contributed to strength to an important degree up to 28 days, but its effect then diminished and became zero or negative at one and two years.

(c) The role of tetracalcium alumino ferrite (C_4AF) is not so clearly indicated. It contributed to some extent to tensile strength after one day and to compressive strength of plastic mortars of 1:2.75 mix and of concrete at the 28-day and later periods.

2. *Contraction in Air:*

C_3S and C_2S contributed, in approximately the same degree, to the total contraction but to a much lesser degree than C_3A, and to a somewhat greater degree than C_4AF.

3. *Resistance to Sulphate Solutions:*

(a) Of the four major compounds, C_3A appeared to be the only one that reduced the resistance of mortar and concrete to 2 per cent solutions of magnesium and sodium sulphate. However, specimens exposed for a period of three years showed good resistance to the magnesium sulphate solution when the C_3A content of the cement did not exceed 6 or 7 per cent, and to the sodium sulphate solution when it did not exceed 6 to 10 per cent.

(b) In general, there was a noticeable decrease in the expansion of cylinders stored in the sulphate solutions with decrease in the C_3A content of the cement. Where the C_3A content was 7 per cent or less, the expansions during the two years covered by the observations were not excessive. The expansion in water also tended to decrease with decrease in C_3A.

4. *Resistance to Freezing and Thawing:*

On the basis of the expansions and losses in weight exhibited by concrete cylinders subjected to an accelerated freezing and thawing test, the higher the total of C_3S and C_2S in the cement, the greater the resistance of the concrete. On the other hand, the greater the C_3A and C_4AF, the lower the resistance. The effect of C_3A in lowering the resistance was indicated to be considerably greater than that of the C_4AF.

Railroad bridge with parallel chord trusses of reinforced concrete spanning the Canal de L'Ourcq at Paris

M. Le Port, Engineer of Ponts et Chaussees; *Les Annales des Ponts et Chaussees*, May-June, 1934, III, p. 400. Reviewed by B. Moreell.

This article describes the details of a reinforced concrete single track railroad truss bridge, with span of 27.82 m. (91.5 ft).

The bridge consists of two Pratt trusses, the bottom chords of which are connected at the panel points by cross beams built monolithically with the bottom chords. There are nine panels of 3.091 m. each. The bridge is built on a skew of 8 deg., 45 min., the cross beams making this angle with the axis of the bridge. Under each rail there is a longitudinal girder, framing into the cross beams. The deck of the bridge is a slab 4.8 in. thick. The cross-ties are set in ballast resting on the deck. The thickness of ballast is not stated.

Some interesting features of this structure follow:

A Howe truss was first considered but was rejected because the length of embedment of the vertical bars would be insufficient without a great increase in the thickness of the lower chord. The Pratt type was adopted, the tension bars in the diagonal members being extended, in the form of a sine curve, into the lower chord and parallel to the axis thereof.

The reinforcing steel at the lower panel points is greatly congested. In order properly to place the concrete at these places, it was necessary to vibrate the forms and the reinforcement *simultaneously*. In addition, steel rods were inserted between the reinforcements and the former were vibrated by small air hammers. As an additional precaution, small tubes were cast into the concrete at all panel points. When the work was completed an attempt was made to inject grout under pressure into these

tubes, but the amount so injected was inappreciable, indicating that the voids surrounding the reinforcement had been well filled.

The lower chord was 21 x 27 in.; the upper chord 21 x 24 in.; the cross beams 24 x 25 in.

Splices in tension steel were made by lapping, without hooks. The upper and lower chords were heavily reinforced with lateral steel. In calculating the anchorage of tension steel in the diagonals, account is taken of the frictional resistance of the steel-concrete contact surfaces in the sine curves, but details as to how this frictional value was computed are not given.

The trusses are simply supported, one end resting upon a convex seat of reinforced concrete, the other end resting on a short post (1.12 m.) with both ends convex. This post acts as an articulation. The lower end of the post rests on a concrete seat and is held in place by a short steel bar. This arrangement permits changes in length of the truss without introducing excessive stresses.

Upon completion of the work it was tested under a static load, which would give a bending moment at the center of span equal to the design bending moment. The actual deflections under this load were less than the calculated deflections, even though account was taken of the tensile value of the concrete. An interesting feature is that, although both trusses were under the same load, the deflections of one were a little over half those at the corresponding points of the other truss.

The working stress in the concrete was 1060 p.s.i.

Setting of portland cement—chemical reactions and the role of calcium sulfate

PAUL S. ROLLER, Nonmetallic Minerals Experiment Station, U. S. Bureau of Mines, New Brunswick, N. J., *Industrial and Engineering Chemistry*, Vol 26, June, 1934, p. 669-677.

Reviewed by ROY N. YOUNG

This paper is the first of a series connected with the problem of the utilization of natural anhydrite as retarder for the set of portland cement. The author describes fully the methods and tests employed for the purpose of evaluating the reactions which actually take place when portland cement is mixed with small proportions of water as in general practice. A study was made of the liquid phases of cement pastes under different conditions in conjunction with measurements of time of set. The liquid phase was extracted from the plastic mass by squeezing under pressure of 280 kg. per sq. cm. The solutions thus obtained were analyzed to determine the various components dissolved from the cement. By a study of the results certain reactions were indicated under the various sets of conditions used, and through co-ordination of these conditions with setting time results, essentially the following conclusions were reached.

Gypsum added to clinker may, on dissolving, react with alkali in the liquid phase, to form calcium hydroxide in solution, according to the equation

$$CaSO_4 + \begin{matrix}2(KOH)\\(Na\ \ )\end{matrix} = Ca(OH)_2 + \begin{matrix}(K_2SO_4)\\(Na_2)\end{matrix} \quad \ldots\ldots (1)$$

and to lower the pH. Also, under certain conditions, it may react to form calcium sulfoaluminate as in the equation

$$\begin{matrix}(3CaO.Al_2O_3\ (s)\ \ \ \ \ \ )\\(3CaO.Al_2O_3.XH_2O\ (s))\end{matrix} + 3CaSO_4 = \begin{matrix}3CaO.Al_2O_3.sCa-\\SO_4.XH_2O\ (s)\end{matrix} \quad \ldots\ldots (2)$$

Calcium hydroxide delays the hydration and set of tricalcium aluminate and retards the set of portland cement. The retardation of the set by gypsum is interpreted as being due to its formation of calcium hydroxide in solution. The retarding action is accomplished by the reaction of calcium hydroxide with tricalcium

aluminate and the formation of hydrated tetracalcium aluminate, in accordance with the equation

$$\left.\begin{matrix}(3CaO.Al_2O_3\ (s)\qquad)\\(3CaO.Al_2O_3.XH_2O\ (s))\end{matrix}\right\} + Ca(OH)_2' \quad 4CaO.Al_2O_3.XH_2O\ (s) \qquad (4)$$

This hydrated tetracalcium aluminate is insoluble and forms at the surface of the tricalcium aluminate, which hinders further hydration of this compound, thus affecting retardation of the set. In a solution containing a high concentration of calcium hydroxide and a low content of alumina, typical of the liquid phase in normal cement pastes, the hydrated tetracalcium aluminate is the stable phase.

Another source of calcium hydroxide for reaction with tricalcium aluminate is in the hydrolisis of tricalcium silicate, which in itself is a retarder for tricalcium aluminate. The action of gypsum in the presence of tricalcium silicate may then serve only to supplement its retarding effect.

The alkali compounds in cement dissolve rapidly and in solution react with calcium hydroxide to precipitate the insoluble calcium salt. At low concentrations of calcium hydroxide, insoluble calcium sulfoaluminate is formed and in the case of extensive formation of this compound, quick setting of the cement may occur. Therefore it appears that the formation of sulfoaluminate may be a direct cause of quick setting as well as being indicative of insufficient concentration of calcium hydroxide for the formation of the retarding compound tetracalcium aluminate.

Photo-elastic research on slab-and-beam floors

M. Suquet, (Inspector-General of Ponts et Chaussees), *Les Annales des Ponts et Chaussees*, Vol. II, No. IV, July-Aug., 1934. Reviewed by B. Moreell.

There is a considerable disagreement in current practices with respect to the treatment of slab-and-beam floor design and particularly in the assumptions relative to the degree of participation of the floor slab in the resistance of the beams. Procedures for determining the effective width of the T-beam flange vary and are not in agreement as to which factors are significant.

The principle factors which might influence the effective width of flange are:

(a) Span of the beam
(b) Spacing of floor beams
(c) Width of beam
(d) Depth of beam
(e) Thickness of slab.

The majority of regulations in current use contain empirical formulas which limit the effective width of flange as a function of factors (a) and (b). The latest Swiss regulations utilize factors (a), (b), (c) and (e). The German regulations utilize factors (b) and (d) only.

On account of these important differences in treatment and in view of the very limited experimental data available on this subject, the laboratory of the Ecole Nationale des Ponts et Chaussées undertook, in 1934, an investigation by the photo-elastic method of the distribution of stresses in the slab and in the beam of a floor of this type. The particular object of this investigation was to determine to what extent the effective width of flange is affected by variations in (a) the thickness of the slab and (b) the depth of the beam.

Before describing the experimental work and the results obtained, the author reviews briefly the principles of the photo-elastic method of stress determination. This discussion is of special interest in that it contains a brief statement of Mesnager's method of determining the actual principal stresses at any point. The difference of

the principle stresses is first determined in the usual manner. The sum of the principle stresses is then determined by measuring variations of the thickness of the specimen by optical means. This is of particular interest in view of the discussion in the Sept.-Oct., 1934, JOURNAL* on Doctor Westergaard's paper "Stresses at a Crack, Size of the Crack and the Bending of Reinforced Concrete."†

The model consisted of a sheet of glass of uniform thickness, 106 cm. wide and 340 cm. long. Along each of the long edges of the sheet there was a beam (also of glass) 6 cm. wide. There were 14 models, distinguished by differences in slab thickness (2 mm., 4 mm., and 6 mm.) and in depth of beam (10 mm., 20 mm., and 30 mm.).

In the first series of tests the models were freely supported and loaded with a concentrated load at the center. In the second series the concentrated loads were applied at the quarter points.

To obviate the necessity of making corrections for residual stresses in the glass which were not removed by annealing and, also, to increase the range of loading which was very limited on account of the low tensile strength of the glass, the experimenter, M. Tesar, devised an ingenious method of pre-stressing the beams in compression.

The article discusses in some detail the results of the experiments. The following conclusions were drawn:

First Series: Concentrated load in center of span. 1. For the same depth of beam, the stresses in the slab at mid-span of the panel and in the direction of the beam span are more uniform as the thickness of slab increases. For the same slab thickness, these stresses are more uniform as the depth of beam increases. Even for extremely thin slabs and regardless of the depth of beam, the limitation of the width of compression flange to 0.75 times the center to center spacing of beams is very conservative. This limitation appears in the French Government Concrete Regulations.

2. For sections at the quarter point and near the support the same observations are applicable.

3. The slab over the support and adjacent thereto is subjected to a tensile force in the direction perpendicular to the beams. This effect disappears rapidly and at mid-span it has changed to a compressive force. This would be of importance in considering the end span of a series.

4. With respect to stresses in the beams, while the bending stresses are in approximate agreement with the theory, the shearing stresses are decidedly different. The shearing stresses should be constant from the support to the center of span, with discontinuity at that point. In reality, the shearing stresses are a maximum at the quarter points and zero at the center.

Second Series· Concentrated loads at quarter points.—The results obtained from this series confirm those of the first series. Of particular importance is the considerable difference between the theoretical and actual distribution of shearing stresses.

Charts showing stress variations in the direction of the span and perpendicular thereto are given.

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*This volume, p. 87.

†JOURNAL, Amer. Concrete Inst., Nov.-Dec., 1933, *Proceedings* Vol. 30, p. 93.

Trends in the Production and Use of Various Types of Hydraulic Cements*

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INTRODUCTION

I HAVE agreed to write upon this assigned topic with much hesitancy and after some persuasion, knowing that the fears of our Secretary-Treasurer that I *might* "risk repetition", have a sound basis, for I can confidently state that there will be repetition of thoughts which I have previously expressed. One cannot write on essentially the same topic five or six times and in any possible way escape repetition. Hence, if this dissertation smacks decidedly of being "old stuff," direct your silent or expressed comments to the Program committee and our earnestly striving and most frequently accomplishing Secretary. The author of *Ecclesiastes* was right when he wrote: "of making many books there is no end," but please do not always lay the urge for writing against the author.

Let us review the situation regarding the types of hydraulic cements as it exists in the United States. Every constructing agency of the Federal Government is purchasing portland cement meeting the present Federal and ASTM Standards (Class 1 in a Standard which I proposed in June of last year**).

The Bureau of Reclamation is purchasing a special portland cement of low heat of hardening for Boulder dam (Class 2). The United States Engineer Office is buying in addition to the first of these just cited a special portland cement to resist active SO_3 waters for the Fort Peck Dam (Class 3). Practically every state highway department and many municipalities in the United States have purchased high early strength

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**At that time I distributed a letter to a large number suggesting one specification for five types of hydraulic cement (Portland, Modified Portland, Portland-Pozzolanic, Fused and Natural cements, including hydraulic lime, pozzolanic cements, etc.) to meet one or more of four different classes of service—Class 1, moderate strength demands; Class 2, low heat of hardening; Class 3, high resistance to sulfate-bearing water; Class 4, high early strength.—AUTHOR

portland cement (Class 4). Hence, some departments of the Federal or State governments are using or have used true portland cements of all the four classes which were included in my grouping, and the general consuming public has done likewise.

There is being used in the Shenandoah National Park a cement made of a mixture of portland cement and hydraulic lime. The builders of both the Golden Gate bridge and the San Francisco-Oakland bridge, in San Francisco, are using a portland-pozzolanic cement, a blend of portland cement and artificial pozzolana. The Bureau of Reclamation constructed both the Elephant Butte and the Arrowrock dams of a blend of portland ground with local rock, which might be considered a portland-pozzolanic blend. A United States Engineer has used a portland-artificial pozzolanic cement in part of the sea wall in the Norfolk, Virginia, area. One of the steel-pipe manufacturers is lining steel pipe with a portland cement-blast furnace slag blend. Hence, it would appear that my suggested portland-pozzolanic type of cement has actually been used in all of the four classes excepting the high early strength one.

The type which I have referred to as modified portland cement, and which is a mixture of true portland cements with not more than 5 percent of other materials, comprises the waterproofed cements and those containing such additions as calcium chloride, tannic acid, diatomaceous silica, etc. No example can be specifically cited of the use of these where low heat of hardening or resistance to sulphate waters was desired. But one of the oldest brands of high early strength cement has always had a chloride as an essential part of its composition, and a number of the other brands of high early strength cement contain tannic acid. Portland cements modified by waterproofings or plasticizers have for years been used in numerous cases where cements of moderate strength were needed. Hence, this type of cement has been used in two of the four suggested classes.

The fused or high alumina cements have from their first appearance on the market been sold and used as eminently high early strength cements and for resistance to sulphate waters. The Bureau of Yards and Docks of our Navy Department, has used approximately 8,000 barrels of this cement in one dock in Puget Sound.

The hydraulic limes, natural cements, and artificial or natural pozzolanic cements (Type N of the suggested grouping) have in some instances a most enviable record in the United States—particularly the natural cements, which, as is well known, antedate the portland cements the world over. They have, of course, been used as cements

of moderate strength and for resisting the attack of sulphate water. Their use in the foundation of the Brooklyn Bridge and in the Pennsylvania Railroad Tunnels underneath the Hudson River is well known. They have also doubtless been used in mass construction before we became "low heat cement conscious," where low heat cements are now indicated.

WHY THE DIFFERENT VARIETIES OF CEMENT

It would seem, therefore, that in the United States, either the producer or consumer, or both interests, have advanced to the stage where they definitely recognize, use and demand not only standard portland cement, but a number of cements which it seems sometimes leaves my modest five types and four classes of uses somewhat outdistanced. The reason for these varieties is solely that there is a large and growing intelligent group of users who believe that the conditions of use and of service are such that no one cement can satisfactorily meet all conditions, and a relatively large and growing intelligent group of producers who are mindful of the fact that they are using capital to make money through the medium of producing and selling cement. The producers are therefore ready to make, whenever possible, whatever kind of cement their customers will demand, when by doing so they can obtain adequate monetary returns on their investment.

However, there are some who are not certain that all service demands cannot be met with one hydraulic cement, such as our present portland cement covered by specifications of both the ASTM and the Federal Specifications Board. They cite the many favorable results and the long life which have been obtained by the use of this cement in the United States. Their citations are correct and apropos. But granted that 98 percent of all installations are eminently satisfactory, it must be acknowledged that 2 percent or less of failures can cause many heartaches. Is it a matter though of actual failures which has caused the departure from standard portland cement in the country? Is it not rather a growing realization that a job can possibly be done better and with more satisfaction to the contractor, if not also the owner, by the use of some other type of cement?

NATURE OF CERTAIN OF THE SERVICE DEMANDS

In this question of cements for different classes of service other than those to be met by the ASTM or Federal standards, let us consider first the low heat cements. To most persons it seems that the thought of such cements originated with the designers of the Boulder dam. But that matter has been discussed by dam builders in this country

possibly from the earliest use of hydraulic cements for such purposes. One of the first major discussions of this subject in the United States appears in the 1908 Transactions of the American Society of Civil Engineers as a result of a paper by Gowen entitled, "The effect of temperature changes on masonry." At this time the Bureau of Reclamation, which designed Boulder dam, was but six years old. Further, it seems that the reason for using such cements in Boulder and other dams is largely misunderstood, though the correct reason has been adequately stated before. The type of cement was selected because it developed not only low heat and therefore low temperature while attaining rigidity, but also a total low heat or temperature. It would thus have a low shrinkage due to cooling even from the low temperature attained, which shrinkage by the construction used gave adequate space for grouting between the blocks or prisms of which the dam is composed—the dam is not a monolithic structure. It may be that this cement does have a low coefficient of thermal and humidity change, but that is not sought for in the cement used and some results would seem to indicate that it was not attained. The thought to be particularly remembered is that this cement when used in thin sections will not assure any less shrinkage cracking due to thermal or humidity changes after hardening than a Standard cement. Another almost equally important factor always to be borne in mind is that low rate of evolution of heat is accompanied by a low rate of gain of strength. Hence, during the winter months at Boulder dam it was necessary to use blends of 40 per cent Standard cement with 60 per cent low heat cement to permit maintaining the desired rate of construction.

The cements for resisting sulphate or other aggressive waters are a matter of much discussion now, but again, this discussion has extended back to prehistoric times so far as cements are concerned. One of the first researches carried out by the Portland Cement Association took the form of financial assistance rendered the Department of the Interior of our Federal Government at its laboratory on Young's Old Pier at Atlantic City. Later, Congress transferred this work to the National Bureau of Standards, which published the data obtained as its Technologic Paper No. 12. This work was started about 1908. A perusal of this paper will show that the low alumina cements now advocated for such purposes were under world-wide discussion at that time and four of them were used in that study. The Krupps of Germany were marketing such a cement as Erz-Zement (it was also included in the Atlantic City work), and the first paper given by me before this Institute was on the subject of iron cements (Erz and not Eisen Zements)

at the 1912 meeting at Kansas City. But after all the discussion through all this time, it must be confessed that there are great quantities of concrete in service in sea water and in other aggressive waters which have given most adequate results in service, and which contain cements having higher ranges of alumina content. I have cited before that the Statue of Liberty stands on a foundation containing Dyckerhoff cement, which had an alumina content of 7.1 per cent (or if preferred, a tricalcium aluminate content of 12) and no fault can be found with its condition. Yet there have developed elsewhere sufficient annoyances to cause many to seek a more resistant cement.

So far as cements of the portland type are concerned, the sulphate-resisting qualities are being sought through changes in chemical composition, which will result in lower calculated amounts of tricalcium aluminate. It is now about 45 years since Candelot suggested the formation of calcium sulpho-aluminate to account for the decomposition of cements in sulphate-bearing waters. Somewhat less than 20 years later the Krupps presented their Erz-Zement, and about the same time Robert W. Lesley, at his new mill at Norfolk, Va., began making a cement containing about 7 per cent of iron oxide and 3.5 per cent of alumina. A plant was also built and operated at Jamestown, Va., making a similar cement. Even before this, the Yankton Cement Co., at Yankton, S. D., had been making a cement containing about 7 per cent of iron oxide and a like amount of alumina. This cement had been used considerably by the Bureau of Reclamation in its projects in that section. I have been told also that the old plant of the Ironton Cement Co. of Ironton, Ohio, now operated by the Alpha Portland Cement Co., at times made cement containing as much as 12 per cent of iron oxide. But gradually the production of these cements was discontinued, although another owner operates the Norfolk plant making a cement in which the amounts of iron oxide and alumina are about equal; however the total is considerably less than that used by Lesley. The reason for this stopping of production of high iron cements was troubles in burning and very slow hardening of the cement. With the present better-trained plant technicians, doubtlessly the first trouble is now of no concern, and with the finer grinding of cement and a better balancing of the components other than alumina and iron oxide, the excessive slow hardening has been overcome. Consequently, it would seem that if one is certain that he can obtain better results with special cements of the portland cement type to resist sulphate waters, they should be insisted upon, and it would further appear that there are plenty of producers who will furnish them.

The third special type of cement now in demand is one developing its strength very rapidly. This demand rather strangely seems to be more recent than any of those which necessitated a departure from the Standard cement. One would have thought that when the rapid-hardening portland cements replaced the slow-hardening natural cements in this country the logical development would have been to have increased the rate of hardening to the maximum. But it was not until the rapid-hardening but slow-setting high alumina cements appeared on the market that the industry was aroused to the possibility of accomplishing equal results with portland cement.

There followed the introduction of double burning which had been used in France for a decade at least previously. Changes in composition, finer grinding of raw materials and clinker, sometimes lighter and sometimes harder burning, less or no quenching and other plant kinks followed, which finally led to cements having such high early strengths that users believed the cement Utopia was at hand. The reaction was almost immediate and of a nature that illustrates the lack of understanding of what portland cement is and what it does when hardening. It seemed to be believed that these high early cements would have not only a high early rate of hardening, but that this rate would continue in a manner like that exhibited by Standard portland cements. Consequently, when it was noticed that frequently the 28 day strengths were but little above the 3 or 7 day strengths, and that at six months or one year many Standard portland cements had strengths equal to or better than some high early strength cements, there was exhibited a chagrin which at times amounted to annoyance and disgust. In a concrete composed of a well made cement and certain proportions of aggregates and water one can hope for a certain maximum strength; by changes in the methods of making the cement and/or the composition, one can, by using the same proportions of aggregate and water change materially the time at which that maximum strength may be attained, but in many cases the maximum will be changed relatively little. This statement does not carry with it the thought that ultimately all cements in the same concrete will develop the same strength, but only the thought that plant composition and manipulation in the production of a high early and a Standard cement at any mill may more materially change the rate of attaining strength than the maximum strength. The opposition on the part of the producers to meet the demands of consumers for a high early strength cement seems to be strikingly less than in the case of the other classes.

NATURE OF CERTAIN LESS EMPHASIZED PROPERTIES OF PORTLAND CEMENTS

From the discussion so far, it seems that the demands of consumers and the willingness of producers to meet the demands will justify the existence of cements to meet four classes of service conditions: (1) cements of moderate rate of strength gain; (2) low heat of hardening cements; (3) cements of high resistance to sulphate waters; and (4) cements of high early rate of strength gain. With the exception of the low heat cements, the discussion so far has been developed largely without much consideration of how often the development of one class of desired properties might carry with it also a major development of other less desired properties. To go into a discussion of the many properties of cements and how they may be modified by production procedure and their relative importance in different kinds of work, would lead into a decidedly long and tiresome dissertation. But this question is important and must be discussed, though briefly. The discussion will concern such properties as volume changes, water requirements for attaining any desired consistency, water-retaining characteristics, ability to produce easily handled concrete, and durability to weathering in the presence of aggressive salt solutions.

Since the amount of water required to produce the hydration accompanying maximum development of strength is much less than that used in dry mixes of concrete (somewhat less than 18 pounds of water per bag of cement are needed to produce a decidedly "complete hydration;" this is less than half the amount used in a 4.5 gallons per bag mix, usually considered a very dry mix), it can be realized that a very large part of the mixing water should really be called "placing" water, and is needed to get the concrete to where it is desired that it should harden. We are, therefore, attaining a strength less than the potential strength and in so doing also materially affecting adversely the properties of volume change and durability. If the composition of a cement or its physical nature can be so modified that the water required for the consistency needed for a certain condition of placing is reduced, we are approaching the potential strength, increasing the durability, and reducing the volume change. In this respect the trend of specifications towards fixing the same cement-water ratio for all cements for testing purposes is at fault. It expedites and renders the testing relatively easy, but penalizes the cement which truly requires less water, by forcing the use of unnecessary water with consequent poor test results, and enhances the test results of cements which actually require more water. It is another example of assuming that all

cements are alike and that they should be so tested. It removes from the manufacturer's production improvements agenda that very important one: the marked desirability of making a cement which will require less water for placement than the one he is now making.

The water-retaining characteristic of cement is an item of more importance than is generally realized and acknowledged. In some cases the retention of water is a very welcome property, but in others it is undesirable. If one is constructing highways or placing cement in intricate, slowly absorptive forms, the cement then used should have maximum water-retaining qualities. When it is of such a nature that water readily separates on standing or in working in the forms, segregation and surface scaling, particularly in highways, ultimately result, but if one is placing concrete in absorptive forms, then the ready separation of water is desirable. In the former case there may also result large or excessive shrinkage on drying, poor strength and durability, particularly if the cement also requires a large amount of water to obtain the needed consistency for placement. It might be well at this time to emphasize that the cement-water ratio which determines the properties is not the ratio at the time of placement, but that at the time of high rate of hardening. Concrete can be placed extremely wet and can be made to give most excellent results in all cases if the water alone can be removed without segregation before the rate of hardening becomes such that some degree of rigidity has been attained.

The property of cements of producing easily handled and placed concrete, (or if a poor definition is acceptable, workable concrete) without the use of unduly large amounts of water, is one which has been too infrequently emphasized. Cement is the plasticizing agent in concrete composed of water, cement and aggregates. All the other ingredients lack this property. To hope for truly good working properties in concrete they must be sought for in the cement. While one cement may have enhanced qualities in this respect, another may be decidedly lacking. The reason for this is not definitely known. Increased fineness of any cement will increase its plasticizing properties; so will certain changes in composition, such as increased alumina or lime, but in this respect, as in certain others, enhancing one desired property may lead also to increasing a less desired one. We can hardly increase the alumina much to obtain good working qualities if we are limited by chemical requirements as in the proposed SO_3-resisting cement. To obtain maximum workability with minimum effort, nothing is so outstanding as one- or two-tenths of a per cent of a stearate ground with the cement, although it has the unfortunate property of

acting as a frothing agent, and hence is most effective in incorporating air into concrete. Long mixing of concrete, or mixing, allowing the mix to stand, and mixing further also develops marked workability, but if this is accompanied by the use of more water, then beware of all the effects of a too low cement-water ratio.

Resistance to weathering due to freezing and thawing, to marked thermal changes or to attack of aggressive salt solutions, has always been an item of much importance. The percentage of failures is not large, but the number is sufficient to have aroused much concern. It is granted that faulty concrete making has been mostly the cause of failures, but we cannot for such reasons leave the cement out of the picture in securing more enduring concrete. Whether we should go to the extent of limiting the composition of cement, hoping thereby to obtain the desired physical property, or develop tests indicative of endurance is actually the crux of the situation now. Until the last two or three years the producers were unitedly against the use of chemical analyses for developing ratios of chemcial components or calculating assumed composition, and strongly opposed the efforts of some rugged individualistic consumers in this respect. But suddenly the producers have changed their attitudes and have "climbed over" or more properly crawled underneath the dividing fence. It is true that the individual compounds in cements have characteristic and, in some cases, distinctly different properties, but we are not certain of the identity of the compounds other than those formed, at equilibrium, of the silica, alumina, iron oxide, lime, and sulphuric anhydride, and are not too certain of the physical properties of these, or that equilibrium has been attained. We are entirely uncertain of the compounds formed by the minor constituents, the oxides of sodium, potassium, phosphorus, titanium, manganese, etc.

INADEQUACIES OF PREDICTING PHYSICAL PROPERTIES FROM CALCULATED CONSTITUTIONS

Without discussing whether the procedure of specifying the percentages of the four major compounds, calculated according to certain assumptions, has more merit than that of specifying the ratio to be maintained by certain oxides known to be present (although the latter method has always been considered of little merit in this country), it is decidedly proper to consider upon what hypotheses the calculations made to determine the amounts of these compounds are based. Judging from the queries that have been made, it would appear that the calculations are carried out as given in certain specifications and little thought is given to their basis until some troubles are encountered.

Possibly the most important among the premises used is the assumption that the cement is composed only of silica, alumina, iron oxide, lime and SO_3, and that the other oxides should be ignored. Of next importance is the assumption that when equilibrium has been attained (and we are not certain of this in commercial cements) the silica is present either as tricalcium or dicalcium silicate or both, the iron oxide as tetracalcium ferro-aluminate or calcium ferrite, the alumina as the tetracalcium ferro-aluminate and tricalcium aluminate, and the SO_3 as calcium sulphate. It is of no use to inquire in making the calculations according to the specifications giving them, how the ferrous oxide, the oxides of sodium, potassium, manganese, phosphorus, titanium, the 4 per cent ignition loss permitted by most of our stand ards, etc., are concerned in these calculations. To date, the specifications, where limits are placed upon the calculated compounds, ignore these without calling particular attention to their having been ignored. Having ignored their presence, these specifications tacitly, through their failure specifically to note otherwise, assume that if present they neither enter into the combination with the five oxides mentioned nor affect the calculation of the compounds assumed to be present.

Further, the possibility of changing the properties of the compounds by quenching, heat treating, etc., is ignored. There is evidence to show that quenching from certain temperatures to others or heat treating at certain temperatures will materially modify the properties of the finished cements, and it would naturally be inferred it must have also changed either the kind of compounds assumed as present or changed their individual properties.

To illustrate what might happen in one respect—we have recently been interested at the National Bureau of Standards in the question of the amounts of ferrous oxide in cements. We have found that one mill's product on a number of occasions has had 0.7 per cent of ferrous iron present and a total ferric oxide content of approximately 3 per cent. (The latter figure includes the ferrous oxide as ferric oxide according to usual practice.) The calculations made according to the assumptions, with an approximate percentage of alumina of 6, give a tricalcium aluminate content of 10.5 per cent and a tetracalcium ferro-aluminate content of 9 per cent. When allowance is made for the ferrous oxide these figures become 13.5 and 4.5, respectively, and it leaves us uncertain as to what to do with the ferrous oxide or how it affected the amount or nature of the other major constituents. For interesting studies on impurities in excellent large crystals of tricalcium silicate, reference should be had to recent papers by Andersen in the

United States and Guttman in Germany.[1] Anderson refers to clear crystals of this compound free from inclusions, but containing ferrous oxide in solid solution and showing after heating to 900°C lamellae of dicalcium ferrite, and after heating to 1,000° inclusions of dicalcium silicate resulting from the formation of the dicalcium ferrite. He also found crystals of tricalcium silicate containing, by chemical methods, 14 per cent of impurities and by the microscopic method 12 per cent of solid solution. Guttman's report corroborates these findings.

Another illustration of the inadequacy of calculated compounds is furnished by the following: The Bureau of Reclamation has called attention to the fact that each one per cent of ignition loss in excess of an initial one per cent will reduce the heat of hardening 12 per cent. By aging or prehydration to the 4 per cent ignition loss allowed by present specifications, the heat of hardening can therefore be reduced about one-third, an amount sufficient to reduce many standard portland cements to the category of low heat cements. By prehydration we have made at the National Bureau of Standards certain cements of high tricalcium aluminate content pass the Merriman test for sulphate resistance, although before the steaming used to increase ignition loss they had failed. Now, those who are using standards limiting the calculated composition in lieu of physical requirements to predict the desired service, are limiting themselves to a restricted group of cements and tacitly denying that another group might equally well meet their demand, and further denying the usefulness of physical tests.

BACKGROUND OF SOME OF THE OTHER THAN PORTLAND HYDRAULIC CEMENTS

While in this country we have just become what might be called "class conscious," that is, acknowledging that there are different kinds of work and service which demand different cements, although insisting that all cements should be of the portland type, there are other countries where they are and have been "type conscious," and this state of mind seems also to be slowly developing in this country. It is claimed now that with a few exceptions there is not much of a background for the other types in this country. Some may even question the history of some of the exceptions, other than the natural cement, which, as is well known, was made in this country before portland cement and is still being made. However, the facts are that slag cements were made for many years in the United States and to the best of our knowledge are still being made in two mills. In the studies

[1]Properties of tricalcium silicate from basic open-hearth slags, Andersen and Lee; *J. Wash. Acad. Sci.*, July, 1933. Significance and fine structure of tricalcium silicate from the technical cement viewpoint, Guttman and Gille; *Zement*, March, 1933.

of concrete in sea water at Atlantic City, above referred to, four slag cements were used. The predecessor of the Universal Atlas Cement Co., the Illinois Steel Company, made slag cement, and at one time called its product "Steel Portland Cement." A group of portland cement manufacturers seemed to have thought that if the producer of that brand wanted to name the product "Steel Portland," he should at least spell the word with an "e" and an "a" rather than double "e." At least litigation was originated which resulted in a change of brand name and in the production of true portland cement. The blends of natural and portland cements which are of some concern now were once the industrial stepping stones from the natural cement to portland cement. Manufacturers of the former "improved" their product by grinding it with portland clinker. The improvement continued until the product was finally all portland. Two of these improved hydraulic cements were used in the Atlantic City studies and one of them was used in the construction of the viaduct over which trains used to enter the Broad Street Station, in Philadelphia. Hence, it would seem that some of these other-than-portland cements do have some little history in the United States, aside from their present use to which reference was made earlier in this paper.

But if one leaves this country and goes to Europe, it will be found that all the various and many types of cements do have a background of usage which in many respects completely overshadows that of the portland type. Natural pozzolanas, with and without lime, have been used almost as far back as the history of construction extends and are still being used. Naturally occurring products rendered pozzolanic by heating or other procedures have a like history. The cements made in a small part or largely of such artificial pozzolanas as metallurgical slag have been used for years to an extent exceeded only by the true portland. The fused or high alumina cements, although patented abroad about the same time as in the United States, got into production and use a decade before they appeared on the American market in any quantity. In certain countries the portland cements have always been sold in a number of grades, such as slow, medium and quick-setting; moderate, high and early strength, etc. Blends of portland cement with various materials have been available also.

The continued use of the varieties abroad would seem to offset rather positively the argument that since, with the exception of the portland cements, they do not have a history in this country, they should be looked at only with a very negative attitude. But can all conditions abroad be so different that we cannot assume that the results obtained

there can be applied directly here? It would appear that they are not different and hence it can be assumed that the various cements used there would give just as good results if intelligently used here. And are not our workmen as intelligent as those over there? I think all will agree that they are, though many feel that foreign designers are more to the fore, particularly in daring to do new and different things. Possibly the designers are also instrumental in daringly fostering the use of the more varieties of cement abroad.

SUMMARY

Summarizing, there can be no doubt that the trend is decidedly away from one standard for all uses. The demand is for the use of more types and more varieties within any one type depending upon the intended use. The consumers in most cases are to the fore in these demands. On the whole, the producers are still reluctant to meet them. So far there has been no demand for a cement which does not have a sufficient history to show adequately and clearly how it should be used and what the results will be when properly used.

The general significance of the trend and its final outcome will be without any question better concrete. The producers, through the most excellent work of the laboratory of the Portland Cement Association, have shown how concrete may be bettered and also harmed when the water, aggregate, temperature, methods of placement, etc., are variables and the cement a constant. The results have been in a striking number of cases accepted and applied, but there have been a few, and the number is not confined to the consumers alone, who have refused to accept the cement as a constant, and are sure it should be considered just as much a variable as any of the other components of concrete. They are therefore studying them and picking out the one best suited for the purpose at hand. About twenty years ago I was associated in the testing of some large fabricated steel columns to determine whether or not it would be cheaper in a certain bridge to use a certain steel of high tensile and elastic properties at a higher cost per pound than to use the commonly accepted carbon steel at a lower unit cost. The tests showed the alloy steels would permit a lighter section and a bridge at lower cost. Now alloy steels are commonly used in large steel structures. A large concrete bridge has recently been completed in France (St. Thibault on the Loire) in which there was used the high early hardening fused or aluminate cement, because the designer found by tests that by so doing he could use lighter sections than if he had used the cheaper but lower testing portland cements. The difference in the cost of the concrete was such as to

more than make up the difference in the price of the cement. The higher priced cement yielded the bridge of lower cost. So far as the designer and consumer attitude is concerned, it is rather certain that there will be far more intelligent use of cement and this will carry with it the insistent and increasing demand for varieties.

It will not necessarily follow that all producers will make all of the demanded varieties. There was a time when a few manufacturers were in the enviable position of being able to sell their standard portland cement at a slight premium because there were enough consumers who felt that their output was sufficiently outstanding to justify the extra cost. If any manufacturer can continue to make standard portland cement and dispose of it at a rate and a price that will yield him adequate continued returns on his investment, he should bend his efforts to continue in that advantageous position. But if he were assured that the continued demand for another type of cement or a different grade of portland cement would justify his making it, and in so doing he would obtain a satisfactory return on the needed additional investment to produce it, he would be foolish not to take advantage of the opportunity. In the end one can be assured that as soon as the demand for any special cement becomes sufficient to insure the possibility of making money in producing it, some producer will meet the demand, even in spite of all adverse propaganda on the part of other reluctant producers. It may be that many consumers drifting away from standard portland cement do not produce any convincing arguments why they should have another cement, but it is equally true that sales-combative producers do not advance any more convincing argument why the consumer should not have what he wants.

CONCLUSIONS

Finally however, bear in mind that even many times the number of varieties of cements now available in the most prolific countries will not bring about the Utopia of 100 per cent initial and enduring success. Nature in its most apt moods never produced any great masses of perfected rock. The best of our sandstone, limestone, marble or granite deposits all yield a quota of material which cannot be used for structural purposes. On the contrary, one can anticipate many heartaches from the too rash maker of concrete who may use a new type of cement exactly as he has been accustomed to using standard portland. Some of these new types of bonding agents may demand decidedly different handling than was used with the standard portland. Even in such a warm climate as that in which Boulder dam is being erected, it was necessary in the cold weather to use a blend of 40 per

cent standard portland with 60 per cent low heat cement to permit the usual speed of construction. Remember, in cements as now made low heat of hardening is synonymous with low rate of attaining strength, and the weaker the concrete the less it is able to withstand such adverse action as freezing. There was cited the case of the French contractor who had applied the high cost alumina cement to the erection of a bridge, but through the medium of lighter sections, used only so much concrete that the final cost of the structure was less than if he had used standard portland. He was aware, however, of another property of the cement, namely, its markedly high heat of hardening which may be so great as deleteriously to affect the strength. Hence, he thoroughly water-cooled his newly placed concrete and when he felt it necessary, he built his heavier sections in two longitudinal parts. Those who are inclined to use not only the high alumina cements but also the very rapidly hardening portland cements should beware of large masses of concrete, unless artificial means of cooling are resorted to. Some types of cement on mixing with water develop plastic cohesiveness to the extent of being sticky; in some cases this property is accompanied by the need of the use of water to the extent that the specific gravity of the paste is distinctly lower than what would be the case with a good paste of portland cement. When such cement is further used with certain fine sands or with concretes that may contain the proper amount (for average portland cement) of sand, the resulting concrete will contain so much water that during the early hardening shrinkage will occur to an extent that marked cracking will result. All such cements should be used with what could be called (with standard portland cements) under-sanded mixes.

These few citations are given to prospective users of other types of cement than portland, solely to emphasize the cautions—first, stop and think; second, after mature thinking make some tests to determine what are the other outstanding properties of the concrete you will make of the special cement, in addition to the ones which have been so impressed on your mind that you are about persuaded to use the special cement. The citations should also serve to emphasize for prospective makers of special cements the caution—don't market of your own volition new types of cements until you know and can advise how your product should be used. This likely will cost you about as much as it has cost you to learn how to produce your new product. And, lastly, there is a special warning to producers of standard portland cement: The various types of hydraulic cements are here and are here to stay,

and there is no use "kicking against the pricks." Accept the new cements as gracefully as possible, make them if money is to be made in producing them; but if not, don't assume any obstructive attitude. Remember, the world moves on and you either keep pace or are lost.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by July 1, 1935.*

High Early Strength Cements in Concrete Products Manufacture*

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Comparative studies of the use of high early strength cements and ordinary portland cements reported in the papers by Timms and Withey on "Temperature Effects on Strength of Concrete"‡ suggested that concrete products manufacturers might use high early strength cements to advantage.

The results of those studies are based on cylinder tests which may or may not be similar to the results where hollow concrete units are used. Detailed information was, therefore, desired on:

1. The relation of strength to age.
2. Relative strengths with varying amounts of high early strength cements.

The desire was to make comparisons under regular plant conditions rather than under laboratory conditions. A preliminary series of tests using one high early strength and one ordinary portland cement gave such interesting results that two additional series of tests covering all of the high early strength cements and portland cements ordinarily available in the Detroit market were planned and carried out in the plant of Standard Building Products Co. in co-operation with the Concrete Products Association of Detroit. Usual plant practice was followed with adequate supervision to see that the tests were comparable. One series of specimens was made October 23, and a second series November 6, 1934.

MATERIALS

Sand was obtained from a local pit and was not dried before using. It had a fineness modulus of approximately 3.80. (See table p. 242.)

The October sand had a moisture content of 2.7 per cent, and the November sand, after heavy rains in the interval, 4.9 per cent.

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‡Journal Amer. Concrete Inst., Jan-Feb., 1934; *Proceedings*, Vol. 30, p. 159, and Nov.-Dec., 1934, Vol. 31, p. 165.

Retained On	October Series	November Series
⅜ Sieve	2.6%	3.6%
4	20.9%	18.7%
8	40.0%	36.2%
14	56.3%	52.0%
28	75.6%	73.0%
48	93.2%	93.8%
100	99.4%	99.4%
Finess Modulus	3.88%	3.77%

The October tests were of two high early strength cements and eight ordinary portland cements. The November series included the same two high early strength cements and the four strongest ordinary portland cements as revealed by the October series.

The mixture was designed to yield twenty block per batch. This meant using approximately 800 lb. of damp sand per batch. It was impossible to get exactly twenty block per batch, but the results herein given have been reduced to a 20-block-per-batch basis. Water was measured in an overhead tank before being dumped into the mixer. Four gal. per batch were added in the October series, giving a total of 6.6 gal. per batch—three gal. per batch were added in the November series, giving a total of 7.8 gal. per batch. For the high early strength cements a total of 8.0 gal. were used in the November series.

In the October tests one series was included to show the effect of using 70 lb. of high early strength cement instead of a full sack of 94 lb. per batch. In the November test series both high early strength cements were included to show the effect of using 47 lb. as well as 70 lb., instead of a full sack of 94 lb. of high early strength cement per batch.

MAKING AND STORING SPECIMENS

The aggregate sliding down a chute from overhead bins was measured by eye, but due to the experience of the mixer man close yields were obtained. Each batch was mixed dry for two minutes and wet for three minutes in a 9 cu. ft. open Blystone batch mixer. The water dropped into the mixer from a perforated one-inch pipe extending the full length of the mixer. Each batch was dropped independently into the hopper above the block machine so that an accurate check could be kept of the yield per batch. The regular operator handled the Anchor stripper machine and averaged eight to ten blows per block. To get truly representative results which would take into account varying conditions during the day such as might develop in the aggre-

gates or in curing conditions, three batches were obtained for each separate condition. One batch was first made of cement A, then one batch of cement B, and so on until all the cements were used; then second batches from cements A, B, C, etc., and finally third batches from these cements.

The block were stacked on curing racks which were moved by a jacklift truck into a low curing room, 8 ft. wide, 50 ft. long and 7 ft. high. Heat was supplied by salamanders. The temperature in the machine room was approximately 50°F, and in the curing room 90°F. The day after the block were made the racks were taken out of doors and the block taken off and stored under an open shed, protected from the elements, until 24 hours before testing when they were moved to a products plant where the Concrete Products Association of Detroit keeps its testing machine. Capping with gypsum plaster was usually done the day before the specimens were broken. The block for the 2-day tests and 4-day tests were taken directly from the curing room to the plant where the block were broken.

Outdoor temperatures for the 28 days following the October series had a mean of 41°F. For the 28 days following the November series the mean was 43°F. Mean temperatures for the five days the 7-day specimens from the October series were outdoors averaged 39°F. Similarly for the November series the mean was 37°F for the five days.

Table 1 shows the results of the October series. *E* and *K* indicate the high early strength cements E_{70} indicates the batch containing 70 lb. of high early strength cement. Table 2 shows the results of the November series.

Tables 3 and 4 show a comparison of the average of high early strength cements (E and K) and the average of the four best ordinary portland cements.

It must be remembered that these tests were made on hollow 8 x 8 x 16-in. concrete block with a net bearing area of 50 per cent of the gross area. As these hollow block are 8 in. high and the outside webs are only 1¼ in. thick, the unit strengths as reported are not the same as they would be if tested in 8 x 16-in. cylinders, as were used in the tests reported in "Further Studies of Temperature Effects on Compressive Strength of Concrete" (Nov.-Dec. 1934) and it should be kept in mind that the American Concrete Institute specifications for hollow concrete block call for a gross area strength of 700 p. s. i. at the end of 28 days or when delivered on the job.

TABLE 1—COMPRESSIVE STRENGTHS OF VARIOUS HIGH EARLY AND ORDINARY PORTLAND CEMENTS

Test specimens 8 x 8 x 16-in. hollow concrete block. Net bearing area 50 per cent of gross area. Specimens made at 50°F, cured one day at 90°F followed by outdoor air storage. Each value is the average of tests of three specimens. E and K high early strength cements.
Compressive strength pounds per square inch gross area.

Cement	2 Days	4 Days	7 Days	28 Days
A	603	793	773	1079
B	561	709	857	932
C	529	733	690	858
D	603	744	861	909
E	1059	1244	1195	1439
F	659	857	903	1073
G	672	847	755	971
H	427	644	781	940
J	667	834	856	1007
K	1064	1075	1265	1245
E^{70}	648	669	800	1108

TABLE 2—COMPARATIVE STRENGTHS OF VARIOUS QUANTITIES OF HIGH EARLY AND ORDINARY PORTLAND CEMENTS

Test specimens 8 x 8 x 16-in. hollow concrete block. Net bearing area 50 per cent of gross area. Specimens made at 50°F, cured one day at 90°F followed by outdoor air storage. Each value is the average of tests of three specimens. E and K high early strength cements.
Compressive strength pounds per square inch gross area.

Cement	2 Days	4 Days	7 Days	28 Days
A	584	711	842	1206
E_{94}	982	1427	1205	1651
E_{70}	801	1282	1038	1418
E_{47}	416	655	672	826
J	498	761	803	1222
K_{94}	1036	1241	1318	1531
K_{70}	663	810	913	1324
K_{47}	386	524	497	658
F	618	783	813	1058
C	596	927	830	1051

As Table 4 which reports the November series includes a wider range of batches using high early strength cements than does Table 3, the following comments are based on Table 4. Fig. 1 is also based on Table 4.

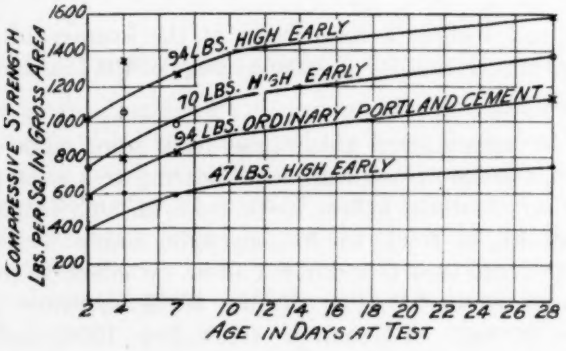

FIG. 1—COMPRESSIVE STRENGTH OF 8 X 8 X 16-IN. HOLLOW CONCRETE BLOCK (20 PER BATCH, 5 MINUTE MIX). MADE WITH VARIOUS QUANTITIES OF HIGH EARLY AND ORDINARY PORTLAND CEMENTS. DATA FROM TABLE 4

TABLE 3—COMPARATIVE STRENGTHS OF HIGH EARLY AND ORDINARY PORTLAND CEMENTS

Test specimens 8 x 8 x 16-in. hollow concrete block. Net bearing area 50 per cent of gross area. Specimens made at 50°F, cured one day at 90°F, followed by outdoor air storage. Each value is the average of tests of three specimens.

Compressive strength pounds per square inch gross area.

October Series	2 Days	4 Days	7 Days	28 Days
Ordinary portland cement				
A	603	793	773	1079
J	667	834	856	1007
F	659	857	903	1073
C	529	733	690	858
Average	615	804	806	1004
High early strength cement				
E	1059	1244	1195	1439
K	1064	1075	1265	1245
Average	1062	1160	1230	1342

TABLE 4—COMPARATIVE STRENGTHS OF HIGH EARLY AND ORDINARY PORTLAND CEMENTS

Test specimens 8 x 8 x 16-in. hollow concrete block. Net bearing area 50 per cent of gross area. Specimens made at 50°F, cured one day at 90°F followed by outdoor air storage. Each value is the average of tests of three specimens.

Compressive strength pounds per square inch gross area.

November Series	2 Days	4 Days	7 Days	28 Days
Ordinary portland cement				
A	584	711	842	1206
J	498	761	803	1222
F	618	783	813	1058
C	596	927	830	1051
Average	574	795	822	1134
High early strength cement				
E^{94}	982	1427	1205	1651
K^{94}	1036	1241	1318	1531
Average	1009	1334	1261	1591
E^{70}	801	1282	1038	1418
K^{70}	663	810	913	1324
Average	732	1046	976	1371
E^{47}	416	655	672	826
K^{47}	386	524	497	658
Average	401	590	585	742

It is noticeable that in most cases the strengths at four days are as high, and in some cases higher, than the strengths at seven days. This is probably due to the fact that the four day specimens were kept inside for three days prior to the breaking, whereas the seven-day specimens were only inside one day before breaking.

RESULTS

(1) High early strength cements gave increased strengths at all ages reported—from 76 per cent at 2 days to 40 per cent at 28 days.

(2) Comparing 70 lb. high early strength cement with 94 lb. ordinary portland cement the increases in strength ranged from 27.5 per cent at

2 days to 21 per cent at 28 days. This checks closely with preliminary tests.

(3) Comparing 47 lb. high early strength cement with 94 lb. ordinary portland cement the decreases in strength ranged from 30 per cent at 2 days to 34.5 per cent at 28 days.

CONCLUSIONS

It must be taken into account that these tests were made on hollow 8 x 8 x 16-in. block with the net area in compression only 50 per cent of the gross area. In considering these values it is well to keep in mind that the American Concrete Institute specifications for concrete block call for 700 lb. gross area at 28 days or when delivered on the job.

Products manufacturers will probably find it of advantage to study the use of high early strength cements in their respective plants. Where it is necessary to put products on jobs quickly, it is apparent that high early strength cements will prove valuable. By taking into consideration comparative costs it may even prove desirable to extend the use of high early strength cements.

It is realized that these tests are not conclusive and it is hoped that further tests will be made by those interested.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by July 1, 1935.*

Effect of Brand and Type of Cement on Strength and Durability of Concrete*

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SYNOPSIS

This paper presents the results of an extensive investigation of the strength and durability of concrete containing eighteen different cements, thirteen of which were standard portland cements and five were high-early-strength cements. Chemical analyses, fineness determinations, and sugar solution tests were made on all cements. The compressive strength on 3 x 6-in. cylinders at ages of 3, 7, and 28 days, on four different concrete mixes was used as a criterion of strength, and the number of repetitions of freezing and thawing of 3 x 6-in. cylinders required to disintegrate the concrete from the same four mixes until a loss in weight of 25 per cent resulted, was used as an indication of durability.

The results showed that the strength as well as the durability of the different standard portland cements varied considerably. In general, the durability was found to be independent of the strength. Both the strength and durability of the concrete seemed to be independent of the chemical composition of the cements tested, indicating that method of manufacturing the cement may have been the most important cause of the variation in the quality.

The special high-early-strength cements gave considerably higher strength of concrete at all ages tested than did the standard portland cements. The durability of very lean mixes was higher for concrete having special cement, but for intermediate and rich mixes there was no difference.

1. INTRODUCTION

Although the problem of disintegration and deterioration of concrete due to exposure is as old as the art of producing concrete, until recently very few data were available for a thorough study of its solution.

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The slogan "Concrete for Permanence" may have contributed to allay the general interest in the problem and may also have delayed the scientific study of the factors which contribute to the production of a most durable concrete structure. However, recent years have seen great emphasis upon the studies of this problem, and it is gratifying to note that the cement industry and its organization, The Portland Cement Association, have through their elaborate field survey* and laboratory experiments contributed generously towards a systematic study of the causes of disintegration. Many of the factors which affect disintegration of concrete have been pointed out by the various studies, but the concrete engineer is still faced with many questions regarding the production of a more permanent structure.

In order to contribute its share towards the solution of this problem, the Fritz Engineering Laboratory of Lehigh University a few years ago, started an elaborate investigation of the strength and resistance of concrete to repeated freezings and thawings. The first part of this investigation dealt with the effect of type of aggregate upon strength and durability of concrete, the results of which were presented in a recent paper.† The second part dealt with the effect of brand and type of cement upon the strength and durability of concrete and the results are presented here. All expenses of these investigations were borne by Lehigh University.

2. ACKNOWLEDGMENT

The Mechanical Engineering Department of Lehigh University cooperated generously throughout the investigation by permitting the free use of its refrigeration laboratory for the freezing of the large number of concrete specimens included in this study. H. R. Nettles former research fellow in civil engineering, and H. D. Comins former graduate student, gave valuable assistance in making and testing specimens, and working up data. Thaddeus Merriman, former Chief Engineer of the Board of Water Supply of the City of New York made chemical analyses and sugar solution tests of all the cements, and the Research Laboratory of the Portland Cement Association computed the chemical compounds.

3. THE PROBLEM OF DISINTEGRATION DUE TO FREEZING

When concrete is subjected to freezing, the free water in its pores changes from a liquid to ice. Concrete in submerged condition has all of its pores filled with water which, upon freezing, changes into ice. Since ice occupies a considerably larger space than water, it will exert

*F. R. McMillan: "Study of Defective Concrete," JOURNAL AMER. CONCRETE INST., May, 1931, *Proceedings*, Vol. 27, p. 1039.

†Inge Lyse and J. M. Holme: "Durability Studies of Concrete and Aggregates," JOURNAL AMER. CONCRETE INST., Nov.-Dec., 1933, *Proceedings*, Vol. 30, p. 121.

pressure on the walls of the minute water pores. As long as all pores are completely filled with water, as in the interior of submerged concrete, the pressure will be uniformly distributed and we have equal compressive stress in all directions, which, for homogeneous materials may be considered as a condition of no stress.* The pores next to the surface are not subjected to this type of uniform stress condition except when the external pressure equals the internal ice pressure. Since the containers in this investigation were made of thin sheet metal, the external ice pressure was very small and the differential in wall pressure for the surface pores becomes important. Ice has about nine per cent greater volume, in unrestrained condition, than the water and since the modulus of elasticity of ice is given at about 400,000 p. s. i., the ice pressure in the pores may attain some considerable values. For an absolutely rigid container the ice pressure would thus be about 36,000 p. s. i. The unequal ice pressure will produce tensile stresses in the walls of pores next to the surface of the specimens. Repeated freezings and thawings will thus set up repeated tensile stresses and the disintegration of the concrete may be considered at least partly as a condition of fatigue failure. The surface will disintegrate first and the progressive destruction will continue towards the interior of the concrete. The strength of the walls is determined by the strength of the cement paste as long as the same aggregate is used. For different aggregates the strength of the walls of the pores as well as the soundness of the aggregate will affect the resistance to freezing and thawing. Both the quality of the cement paste and the type of the aggregate therefore contribute to the resistance of the concrete to these destructive forces. If it were possible to evaluate the size of the pores and the thickness of their walls, the resulting tensile stresses in the walls could be evaluated from the ordinary analysis of stresses in hollow spheres when the pressure differentials are known.

Since the maximum effect of the freezing and thawing results when full pores are surrounded by empty pores, the water line is a particularly exposed place in all concrete structures partly submerged. Disintegration will therefore generally start at the water line and gradually spread out from this elevation.

During the freezing of the specimens there is a differential in ice pressure from the surface of the concrete towards the interior as the freezing progresses gradually from the surface. Similarly a differential will occur when the specimen is being thawed and the transition from ice to water progresses from surface towards the interior. These differentials also contribute to the disintegration of the concrete, and

*A. Föppl and L. Föppl: *Drang und Zwang*, Vol. I, 2nd Ed., p. 42.

the greater the differentials, that is, the greater the rate of freezing and thawing, the greater is the effect upon disintegration. Similar differentials will obtain if some of the pores in the interior of the concrete are only partially filled with water.

A condition which accompanies the physical destruction by freezing and thawing is the so-called "corrosion" or leaching of concrete, which is more a chemical than a physical nature.* This corrosion will particularly accelerate the disintegration at the water line because of the repeated changing of the water at this elevation. No measurement of effect of corrosion was made in this investigation, but no doubt its effect contributed liberally to the disintegration of the specimens. Since all specimens received the same treatment, the results obtained indicated the destructive effect of both the corrosion and the freezing and thawing.

The above statements have been presented to assist in visualizing the problem of disintegration of concrete. The factors which contribute to produce a concrete of high resistance to the deteriorative forces of freezing and thawing are thus made more apparent for direct analysis.

4. OUTLINE OF TEST PROGRAM

The variation in quality of concrete produced with different brands of standard portland cement has not until recently, been given serious consideration. A committee of the American Concrete Institute in 1930 pointed out the great variety in strength results obtained with different brands of standard portland cement and called attention to the lack of information on this particular subject.† Mr. Bates concludes the report with the statement: "There can be no constituent of concrete more worthy of study than the cement—without it there is no portland cement concrete—yet as an essential constituent of concrete it has been studied less than any of the other components."

The effect of the chemical composition of artificially produced portland cements upon the strength and durability of concrete was recently subjected to extensive study,‡ and much study has also been given to blended cements and other special types such as low heat cements, but the more important question of the variation in concrete obtained with present day standard commercial portland cement remains relatively unknown.

*John R. Baylis: "Corrosion of Concrete," *Transactions*, Amer. Soc. of Civil Engineers, Vol. 90, 1927, p. 791.

†P. H. Bates, Author-Chairman, Com. 202: "Variations in Standard Portland Cements," JOURNAL AMER. CONCRETE INST., Nov., 1929, *Proceedings*, Vol. 26, p. 65. (See also: P. H. Bates, "Trends in the Production and Use of Various Types of Hydraulic Cements" in this JOURNAL.—Editor).

‡H. F. Gonnerman: "Study of Cement Composition in Relation to Strength, Length Changes, Resistance to Sulphate Waters and to Freezing and Thawing of Mortars and Concrete," *Proceedings* of the American Society for Testing Materials, Vol. 34, Part II, 1934.

TABLE 1—CONCRETE MIXES

Mix No.	Cement—Water Ratio by Weight	Mix by Weight
1	1.25	1:3.21:4.82
2	1.58	1:2.47:3.70
3	1.92	1:1.97:2.96
4	2.25	1:1.63:2.45

TABLE 2—CHEMICAL ANALYSES BY THADDEUS MERRIMAN

Cement	SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO	SO_3	Loss	Ins. Res.	Total
1	20.35	2.48	7.62	63.24	2.96	1.89	1.27	0.14	99.81
2	21.74	2.03	6.79	63.25	3.62	1.65	0.76	0.12	99.84
3	20.65	2.54	7.41	62.48	2.93	1.77	1.12	0.21	98.90
4	20.85	2.50	6.47	63.10	3.26	1.73	1.19	0.18	99.10
5	22.65	1.93	5.17	63.18	3.74	1.69	1.10	0.13	99.46
6	20.78	2.57	7.53	63.00	2.84	1.66	0.95	0.11	99.33
7	20.76	2.47	7.33	63.26	3.18	1.55	1.45	0.17	100.00
8	20.35	2.43	7.34	63.05	2.97	1.75	1.24	0.14	99.13
9	20.67	2.45	7.28	63.35	2.40	1.63	1.34	0.11	99.12
10	19.97	2.45	7.15	63.42	3.40	1.82	1.23	0.12	99.44
11	20.72	2.46	7.83	63.35	2.51	1.75	0.76	0.14	99.38
12	20.42	2.47	7.93	63.05	3.04	1.88	0.94	0.12	99.73
13	20.05	2.43	7.47	62.91	2.89	1.92	1.55	0.43	99.22
A	19.45	2.56	7.54	63.28	3.05	2.26	1.27	0.11	99.41
B	20.10	2.47	6.98	62.80	3.40	2.19	1.78	0.10	99.72
C	19.05	2.12	7.08	64.93	2.75	2.21	1.08	0.27	99.22
D	4.65	15.21	44.63	35.02	0.19	——	0.62	——	99.70
E	20.07	2.57	7.53	62.50	2.49	2.27	2.60	0.18	100.03

The investigation reported herewith included a study of concrete made with a number of brands of cement. A total of eighteen brands and types were studied, thirteen of which were standard portland cements, and five of which were special high-early-strength cements. All the cements were produced in the Lehigh Valley district, which at one time produced as much as seventy-five per cent of all the cement manufactured in the United States,* and which still produces a large percentage. The cements were purchased through a local dealer who was instructed to obtain the different freshly made samples directly from each mill. The dealer submitted a statement to the effect that the instructions had been strictly followed.

Four different concrete mixes were made with each cement as shown by Table 1. The sand and gravel used came from the Morristown plant of the Warner Co. of Philadelphia. This aggregate had previously been found to give high quality concrete. The cement-water ratios for the four mixes were 1.25, 1.58, 1.92 and 2.25 by weight. Thus mixes varying from a very weak to a very strong concrete were

*Robert W. Lesley: *History of the Portland Cement Industry in the United States*, p. 68.

obtained. Each batch of concrete contained sufficient material for the making of three 3 x 6-in. cylinders for compressive tests at each of the ages of 3, 7 and 28 days of moist curing, and three 3 x 6-in. cylinders for freezing and thawing tests, or a total of 12 cylinders. The specimens which were subjected to freezing and thawing were cured for seven days in the moist room previous to the exposure test. The moist room had 100 per cent humidity and 72°F, ±2°F, temperature throughout the curing period. Since no better equipment was available, the fineness of each cement was determined by screening a sample through the standard No. 200 sieve.

A ten-pound sample of each cement was placed in a tin can and sealed before being shipped to Mr. Merriman for chemical analysis and sugar solution test.

5. CHEMICAL COMPOSITION, FINENESS, AND SUGAR SOLUBILITY

The chemical analyses of the cements were made in accordance with standard specifications* at the laboratory of the Board of Water Supply, New York City, and the results are presented in Table 2. Since all cements came from the Lehigh Valley district the chemical compositions of the standard portland cements would not be expected to differ materially. Table 2 shows that for the 13 portland cements the SiO_2 content varied only from 19.97 to 21.74 per cent, the Fe_2O_3 from 1.93 to 2.57 per cent, the Al_2O_3 from 5.17 to 7.93 per cent, the CaO from 62.48 to 63.42 per cent, the MgO from 2.40 to 3.74 per cent, the SO_3 from 1.55 to 1.92 per cent, the loss on ignition from 0.76 to 1.55 per cent, and the insoluble residue from 0.11 to 0.43 per cent. Except for Cement D which was a high alumina cement, the chemical compositions of the high-early-strength cements did not differ much from the standard portland cements. The only marked difference was the fact that these special cements contained from 2.19 to 2.27 per cent SO_3 whereas none of the standard portland cements exceeded 1.92 per cent. Cement D contained 44.63 per cent Al_2O_3, while all other cements contained less than 7.93 per cent.

The chemical compounds as computed by the Portland Cement Association Laboratory are given in Table 3. These compounds showed greater variation for the different cements than did the chemical analysis. The tricalcium silicate content varied from 38.8 to 49.5 per cent, the dicalcium silicate from 19.9 to 33.2 per cent, the tricalcium aluminate from 10.4 to 16.8 per cent, and the tetracalcium alumino ferrite from 5.9 to 7.8 per cent for the 13 standard portland cements.

*"Standard Method of Sampling and Testing Portland Cement," Amer. Soc. for Testing Materials Designation C77-32.

Of the four special portland cements, A, B, C and E, Cement C showed the high tricalcium silicate content of 62.6 per cent and the correspondingly low dicalcium silicate content of 7.4 per cent.

Table 3 also contains the fineness of the cements as determined at the Fritz laboratory by screening 50-gram samples through the standard No. 200 sieve. This fineness varied from as low as 88.2 per cent to as high as 93.8 per cent for the 13 standard portland cements, and from 96.3 to 99.8 for the four special portland cements. The high alumina cement had a fineness of 93.6 per cent.

TABLE 3—CHEMICAL COMPOUNDS AS COMPUTED BY PORTLAND CEMENT ASSOCIATION LABORATORY, AND FINENESS OF CEMENTS

Cement	C_3S	C_2S	C_3A	C_4AF	MgO	$CaSO_4$	Loss	Total	Fineness
1	42.4	26.3	16.0	7.6	3.0	3.2	1.3	99.8	93.3
2	38.8	33.2	14.5	6.2	3.6	2.8	0.8	99.9	90.1
3	38.8	30.0	15.3	7.8	2.9	3.0	1.1	98.9	91.3
4	46.2	25.0	12.9	7.6	3.3	3.0	1.2	99.1	92.9
5	42.7	32.8	10.4	5.9	3.7	2.9	1.1	99.5	93.4
6	39.2	30.0	15.7	7.8	2.8	2.8	1.0	99.3	91.1
7	42.2	27.7	15.3	7.5	3.2	2.6	1.5	100.0	88.8
8	44.0	25.3	15.3	7.5	3.0	3.0	1.2	99.3	88.4
9	43.8	26.2	15.1	7.5	2.4	2.8	1.3	99.1	88.3
10	49.5	19.9	14.8	7.5	3.4	3.1	1.2	99.4	89.5
11	39.0	30.0	16.6	7.5	2.5	3.0	0.8	99.4	88.2
12	39.0	29.2	16.8	7.5	3.0	3.2	0.9	99.6	91.3
13	44.5	24.0	15.6	7.5	2.9	3.3	1.6	99.4	93.8
A	49.0	18.9	15.7	7.8	3.0	3.8	1.3	99.6	99.8
B	46.1	22.9	14.3	7.5	3.4	3.7	1.8	99.7	98.6
C	62.6	7.4	15.1	6.5	2.7	3.8	1.1	99.3	98.9
D	High	alumina	cement		0.2		0.6		93.6
E	41.2	26.5	15.7	7.8	2.5	3.8	2.6	100.1	96.3

TABLE 4—LIME-SUGAR SOLUBILITY AND ALKALINITY DETERMINED BY THADDEUS MERRIMAN

	Lime-Sugar Solubility			Alkalinity			
				Before		After	
Cement	P.	M. O.	Difference	P.	M. O.	P.	M. O.
1	29.1	40.5	11.4	5.7	—	5.2	8.7
2	26.3	36.1	9.8	3.9	—	2.0	3.7
3	27.1	37.9	10.8	5.6	—	4.7	8.9
4	25.2	34.9	9.7	4.8	4.8	3.3	6.1
5	25.0	32.7	7.7	3.5	—	1.0	2.1
6	25.9	35.5	9.6	5.9	—	5.3	10.9
7	28.7	39.6	10.9	5.0	—	3.5	7.0
8	27.1	37.2	10.1	6.8	7.1	6.9	13.9
9	29.6	41.1	11.5	4.8	4.9	3.2	5.8
10	29.1	38.9	9.8	4.3	4.3	2.1	4.3
11	27.8	38.6	10.8	4.7	4.7	2.9	5.0
12	26.9	37.4	10.5	5.0	—	3.5	7.1
13	29.8	40.5	10.7	5.1	5.3	4.0	7.8
A	32.8	44.9	12.1	5.9	—	4.9	10.0
B	33.2	45.0	11.8	4.4	4.5	2.7	6.7
C	36.2	47.8	11.6	5.4	5.4	3.3	8.5
D	—	—	—	2.5	8.1	0.2	0.4
E	33.3	45.7	12.4	4.6	4.8	2.9	6.5

P. = Phenolphthalein
M. O. = Methyl Orange
} See, DURABILITY OF PORTLAND CEMENT, *Engineering News Record*, January 9, 1930, p. 62.

The results of the sugar solution test are given in Table 4. Thaddeus Merriman has suggested that the durability of cement may possibly be measured by its sugar solubility* and the results are included here to afford a comparison with results of the freezing and thawing tests.

6. DESIGNING AND PRODUCING CONCRETE SPECIMENS

To study the effect of the cement upon the quality of the concrete, it was essential that all other ingredients be maintained as nearly constant as possible. The constant water-content method of designing concrete mixes† was therefore used for all the mixes in this investigation. The trial batch showed that a 1:1½ ratio by weight between sand and gravel would produce a smooth concrete mix and that 39 gallons net water content per cubic yard of concrete would give consistency corresponding to slumps between two and five inches. Consequently the concrete mixes had an aggregate content of 40 per cent 0 to No. 4 sand and 60 per cent No. 4 to ¾-in. gravel. The gravel consisted of 40 per cent between the No. 4 and ⅜-in. sieves and 60 per cent between the ⅜ and ¾-in. sieves. Since the aggregate was used in dry condition, water corresponding to one per cent of its weight was added to provide for absorption. The four mixes with cement-water ratios of 1.25, 1.58, 1.92 and 2.25 by weight are given in Table 1. Since the net water-content was the same for all mixes, the portion of solids in the concrete, which is generally termed the density, remained constant throughout. Thus the only difference between a lean and a rich mix was the relative content of cement and aggregates. The constancy of the water content was considered very essential in view of the effect of freezing and thawing which, as pointed out in the section on Problem of Disintegration, probably to a large extent is determined by the condition of water voids in the concrete.

Standard methods of making concrete cylinders, capping with neat cement paste, moist curing, and testing for compressive strength were followed.‡

7. CONSISTENCY OF THE CONCRETE MIXES

The consistency of the concrete mixes was measured by using the standard slump cone. The results of these measurements are presented in Table 5. It is worth noting that the slump of all concrete batches remained within the desired limit of from two to five inches

*Thaddeus Merriman: "Durability of Portland Cement," *Engineering News-Record*, Jan. 9, 1930, p. 62.

†Inge Lyse: "Tests on Consistency and Strength of Concrete Having Constant Water Content," *Proceedings*, Amer. Soc. for Testing Materials, Vol. 23, Part II, 1932, p. 629.

‡"Standard Methods of Making Compression Tests of Concrete," Amer. Soc. for Testing Materials. Designation C39-33.

TABLE 5—SLUMP OF CONCRETE HAVING CONSTANT WATER CONTENT

Cement No.	Slump, Inches, for Cement-Water Ratio, by Weight				Average
	1.25	1.58	1.92	2.25	
1	3	3½	6*	4	4
2	2½	1½	2½	3	2½
3	3½	2	2½	3	2¾
4	2½	5	6*	6*	4¾
5	2	3	3½	3½	3
6	3	2	4	3	3
7	2½	2½	4	4½	3½
8	3½	3	3	6*	3¾
9	3	3	3	3½	3
10	3	3½	3	3½	3¼
11	2½	2	3	3	2½
12	3	3½	4½	3	3½
13	4	4	3	4	3¾
Average	3	3	3¼	3¾	3½
A	3	3½	2½	3	3
B	3	2½	2½	3	2¾
C	3	2	2½	2	2½
D	5	2	3	3	3¼
E	3	4	3½	3	3½
Average	3½	2¾	2¾	2¾	3

*Fell down, included in average.

with few exceptions, most of which were caused by the falling down of the concrete cone. The average slump for each cement varied from as low as 2½ in. to as high as 4¾ in. with a grand average of 3½ in. for the 13 standard portland cements, a remarkable uniformity. Similarly the average slump for the high-early-strength cements varied between 2½ and 3½ in., with an average of 3. Thus the effect of the brand and type of the cement upon the slump of the concrete was very slight. However, the apparent smoothness and stickiness of the mixes varied considerably for the different cements used.

The effect of the cement content upon the consistency of the four concrete mixes which had constant water content is indicated by the average of all the cements for each concrete mix. For the standard portland cements the slump varied from 3 to 3¾ in. and for the special cements the slump varied from 2¾ to 3½ in. Within the limits of mixes used in this investigation, that is, from a 1:3.21:4.82 mix to a 1:1.63:2.45 mix by weight, the consistency of the concrete therefore remained practically constant.

These results indicate the justification of the constant water content method for designing concrete mixes of constant consistency. They also indicate the advantage in using this method of design where it is essential that only one variable be studied at one time. Water content, consistency, gradation of aggregates, and density of concrete remained constant throughout the investigation.

8. FREEZING AND THAWING TESTS

The freezing and thawing tests were made in the refrigeration laboratory recently constructed by the Mechanical Engineering Department of Lehigh University. This laboratory consists of a cold room approximately 12 x 7 ft. floor space by 13 ft. high, well insulated, and an ante-room, approximately 6 x 7 ft. floor space by 13 ft. high, also insulated, though not so well. Temperatures as low as —20°F to —30°F could easily be produced in the cold room and the concrete specimens were found to freeze solidly when left at this temperature for about sixteen hours.

Three of the 3 x 6-in. cylinders from each mix and for each cement, were removed from the moist room at the age of seven days and placed directly in water in sheet metal containers of overall dimensions, 4 x 12½ x 14 in. The thin sheet of metal would offer very little resistance to the expansion which resulted from the freezing of the water in the container, so that the external ice pressure on the test specimens was practically zero. Thus a large pressure differential would be produced by the ice in the surface pores of the specimen. All specimens were submerged in water at all times after the removal from the moist room, to prevent drying of any pore, the result of which would have caused additional pressure differentials. Only in a completely saturated condition would it be possible to ensure the same pore condition for each test specimen.

The refrigerator was started at about 8:00 a.m. and by about 3:00 p.m. the cold room would have a temperature of nearly —20°F, at which time the metal containers in which the specimens were submerged in water were placed on a platform cart and wheeled into the cold room. The refrigerator continued to run until 5:00 p.m. when it was shut off for the night. The specimens remained in the cold room until 8:00 a.m. the next day at which time they were frozen solidly. The carts were then removed from the cold room to a room in which the temperature was between 70°F and 80°F. Electric fans were placed near the carts to force the air to circulate about the containers. The specimens were completely thawed and had attained a temperature of about 70°F by about 3:00 p.m. and were placed in the cold room for another cycle of test. The rate of freezing and thawing was thus relatively slow, one cycle being obtained in 24 hours.

The specimens were carefully weighed on a gram scale at the time of removal from the moist room, after ten cycles and then after each five additional cycles of freezing and thawing. After each weighing, fresh water was placed in the metal container in order to decrease the

TABLE 6—COMPRESSIVE STRENGTH AT DIFFERENT AGES

Cement	3-Day Strength				7-Day Strength				28-Day Strength				Average
	Cement-Water Ratio				Cement-Water Ratio				Cement-Water Ratio				
	1.25	1.58	1.92	2.25	1.25	1.58	1.92	2.25	1.25	1.58	1.92	2.25	
1	570	1100	1510	1990	990	1640	2330	2870	1450	2580	3390	4640	2090
2	470	890	1470	1840	950	1510	2120	2830	1390	2650	3820	4520	2040
3	420	1010	1450	2020	1010	1730	2460	2760	1490	3160	4110	4970	2220
4	540	1020	1560	2280	910	1760	2600	3700	2120	3190	4270	5550	2460
5	530	720	1180	1860	810	1470	1860	3030	1460	2960	3820	5330	2090
6	570	940	1490	2130	890	1760	2340	3070	1740	3080	4230	4840	2260
7	420	810	1420	1660	730	1660	2300	3180	1490	2670	4330	4430	2100
8	480	860	1490	1970	780	1580	2320	3180	1490	2610	3970	4070	2070
9	410	820	1420	1940	860	1490	2460	3050	1660	3080	3780	4430	2120
10	470	870	1440	1950	860	1570	2260	2990	1420	2870	3510	3930	2010
11	380	840	1360	1980	720	1420	2260	2830	1590	2740	3560	4410	2010
12	530	870	1350	1830	790	1500	2000	2760	1240	2680	3290	4130	1910
13	450	1040	1650	2330	950	1910	2770	4050	1880	3330	4070	5180	2470
Average	480	910	1450	1980	870	1610	2310	3100	1570	2890	3860	4650	2140
A	840	1800	2640	3450	1380	2790	3700	4890	2110	3610	4800	5190	3100
B	1030	2160	3120	3860	1550	3330	4250	5260	2610	4150	5390	5920	3550
C	960	1730	2770	3170	1420	2400	3320	4680	1820	3470	4550	5300	2970
D	1790	3320	4020	4960	1600	3270	4720	5450	1600	2440	4530	5420	3590
E	740	1600	2600	2710	1130	2500	3370	4390	1860	3780	4740	6440	2990
Average*	890	1820	2780	3300	1370	2760	3660	4810	2100	3750	4870	5710	3150

*Cement D not included.

relative effect of change in the chemical composition of the water as produced by the corrosion of the concrete.

The freezing and thawing was continued until the average loss in weight of the three cylinders of each set reached 25 per cent of their original average weight. Since relatively slow freezings and thawings were used, the tests continued for more than two years before all specimens had lost the specified weight.

9. RESULTS OF COMPRESSION TESTS

The average strength of the three cylinders of each kind is given for each cement at each of the three ages of test and for each of the four cement-water ratios in Table 6. To make a more direct study of these results the relationship between strength and cement-water ratios is presented graphically for each cement in Fig. 1 to 6, in which each figure contains the relationships for three cements. It is noted that the results group themselves well along straight lines for each age at test, particularly well, when it is realized that about two months elapsed between the making of the first and the last concrete mix for each cement. The strength for all cements may therefore be represented approximately by the formula $S = A + B\frac{c}{w}$, in which S represents the strength of the concrete, A is a constant indicating the imaginary strength value for $c/w = O$, B represents the effectiveness of the cement, or in other words, the slope of the strength line, and c/w

represents the cement-water ratio in terms of pounds of cement per pound of water, or in other convenient terms. Certain test results will naturally be somewhat off, due to the impossibility of securing ideal uniformity in the process of manufacturing, capping, curing, and testing of the 3 x 6-in. concrete cylinders used. The number of irregular results was relatively low in this investigation as will be seen from an inspection of all the strength diagrams presented in Fig. 1 to 6 inclusive. The straight line cement-water-ratio-strength relationship seems thus to represent the strength of concrete containing ordinary types of both standard and special portland cements. Even the concrete containing the high alumina cement (Cement D) gave approximately straight line strength relationship.

A study of the strength lines in Fig. 1 to 6 reveals that both the inherent strength quality as represented by the constant A in the above equation and the strength effectiveness of the cement as represented by the constant B differ materially for the different cements. There are, therefore, no straight line formulae with definite constants which will represent the strength relationship for various cements. Each cement, due to its chemical compositions and to its method of production, has its own strength quality different from other cements of the same type, and probably also different from time to time in the same mill due to the impossibility of maintaining absolute constancy in the processing. All we can do is to present general strength laws containing factors which may be so selected that they represent the cement under consideration. The straight line strength formula represents the most simple strength law for concrete, namely, the straight line relationship between strength and cement content per unit of net mixing water.

To visualize the variation in the strength of the concrete it is well to study the variation in the constants A and B of the strength law for the different standard portland cements included in this investigation. The equation for a concrete of low three-day strength was:

$$S_{low} = -1300 + 1380.\frac{c}{w}$$

where c/w is the weight ratio, that is, pounds of cement per pound of water.

For concrete of high three-day strength the equation became:

$$S_{high} = -1800 + 1800.\frac{c}{w}$$

Similarly for the seven-day results the strength equation varied from:

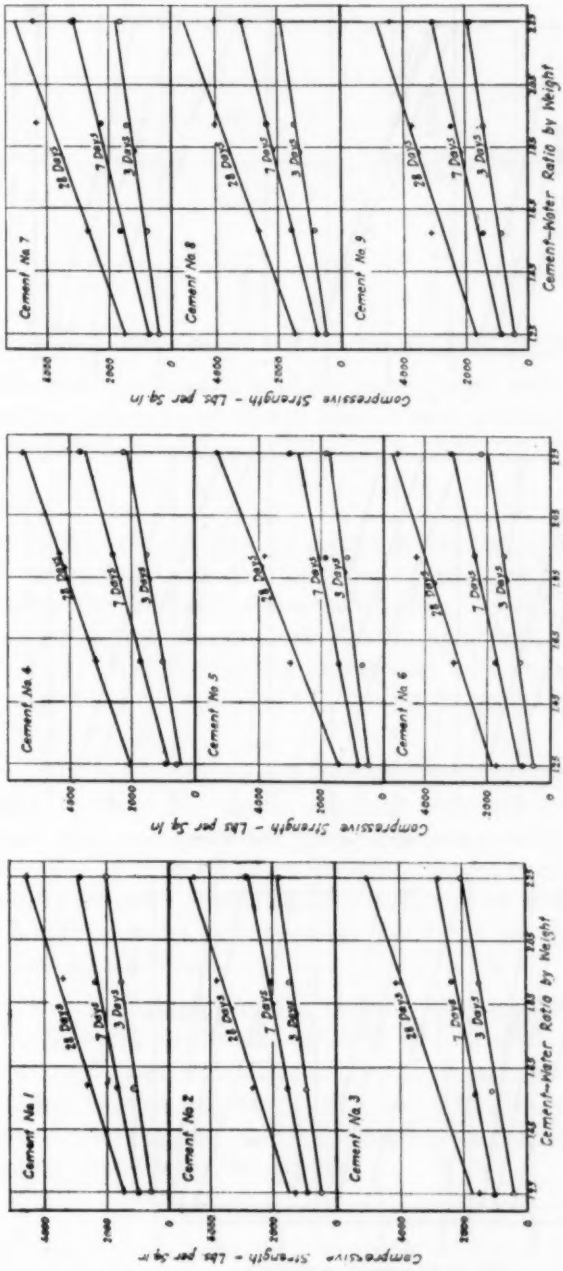


FIG. 1, 2, 3—COMPRESSIVE STRENGTHS OF CEMENTS

FIG. 1—CEMENTS 1, 2 AND 3; FIG. 2—CEMENTS 4, 5 AND 6; FIG. 3—CEMENTS 7, 8 AND 9

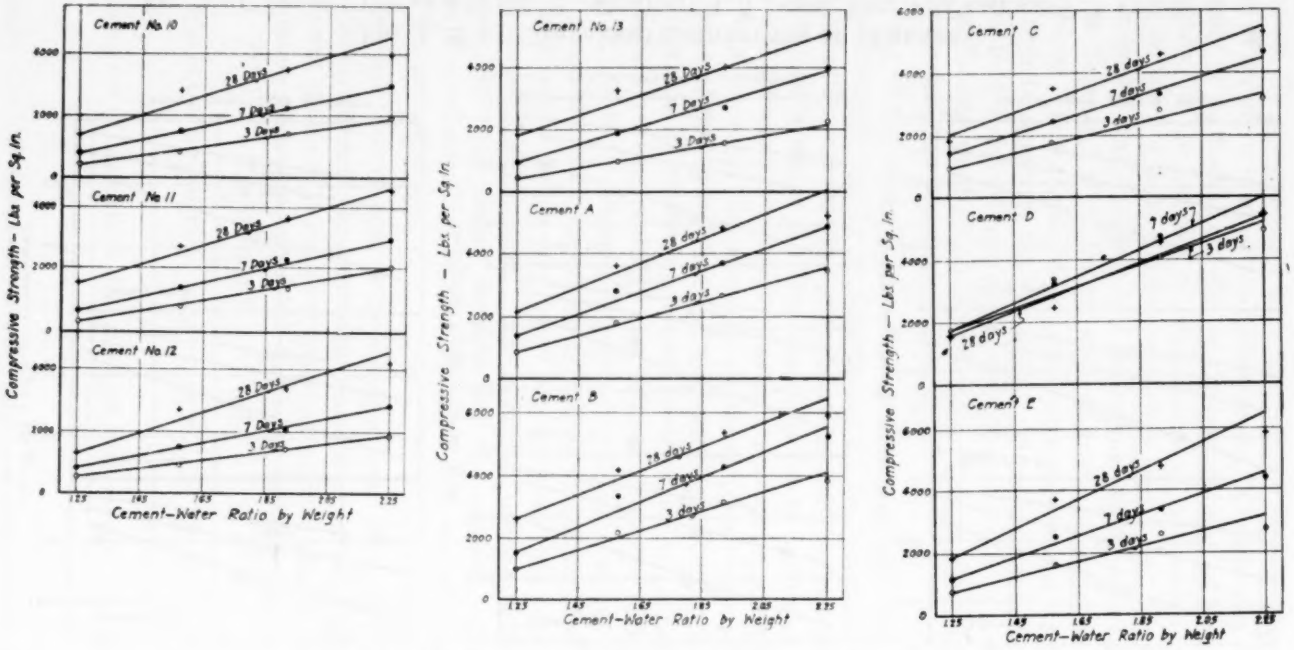


FIG. 4, 5 AND 6—COMPRESSIVE STRENGTHS OF CEMENTS

FIG. 4—CEMENTS 10, 11 AND 12; FIG. 5—CEMENTS 13, A AND B; FIG. 6—CEMENTS C, D AND E

$$S_{low} = -1700 + 2000.\frac{c}{w}$$

to

$$S_{high} = -2800 + 3000.\frac{c}{w}$$

For the 28-day results the corresponding equations were:

$$S_{low} = -2500 + 3000.\frac{c}{w}$$

and

$$S_{high} = -2150 + 3400.\frac{c}{w}$$

Similar variations obtained for the special portland cements.

Since the water content w was the same for all concrete mixes, w might well be incorporated in the effectiveness constant so that the strength formula becomes:

$$S = A + B\frac{c}{w} = A + \frac{B}{w}.c = A + K.c$$

The strength of all concrete specimens containing portland cements increased consistently with the increased length of moist curing. In general, both the inherent strength quality, A, and the strength effectiveness, B, of the cements, varied with age of test. In most cases the slope of the strength line became greater with increased age, indicating that the effectiveness of the cement increased with increased length of moist curing.

The concrete containing special portland cements showed relations very similar to those containing standard portland cements, except that the strength, particularly at early ages, was considerably greater. As noted from Fig. 5 and 6, the strength-cement-water-ratio relationship generally followed straight lines for all ages at tests, and the increase in length of moist curing showed much the same indications as did the standard cements.

For Cement D, a high alumina cement, the straight line relation between cement-water ratio and strength of concrete seems to fit the results fairly well, but the difference in age of test above three days does not seem to have any effect upon the strength. Evidently the three days of moist curing were sufficient for the full development of the maximum strength for this cement.

10. DISINTEGRATION OF CONCRETE

As previously pointed out the specimens subjected to freezing and thawing were weighed after ten cycles and then after each additional five cycles. The variations obtained from these weighings are pre-

sented in Fig. 7 to 14 inclusive, for all cements and mixes used. The plotted values are the averages of the three specimens used as a unit. It is noted that the weight increased during the first ten cycles, indicating that there were either partially empty pores in the concrete previous to the freezing test, although all specimens had been moist cured continually, or the freezing increased the size of the pores. Thereafter there were no further increases in weight, and with additional cycles a decrease would result accompanied by slight surface disintegration. The rate of loss in weight was small to begin with, but gradually increased. When a loss of from two to five per cent had taken place, the rate of loss usually became great, as is indicated by the steepness of the curves.

Fig. 7 to 10 present the results for all cements when cement-water ratios of 1.25 and 1.92 by weight were used, the broken lines representing the former, and the solid lines representing the latter. The cement content is seen to have a marked effect upon the resistance of the concrete to the action of freezing and thawing, the solid lines being an appreciable distance to the right of the broken lines. The individual cements are seen to give considerable variation in durability of the concrete, the number of cycles of freezing and thawing needed for producing a 25 per cent loss in weight being only 88 for Cement 2 and as high as 274 for Cement 7. The concrete containing special portland cements varied from 193 to 305 for cement-water ratio of 1.25. For the solid lines which represent concrete of cement-water ratio of 1.92, the cycles at 25 per cent loss varied from 332 for Cement 11 to 430 for Cement 2, the variation being much less than in the leaner mix. The corresponding variations for the special portland cements were from 335 to 389 for cement-water ratio of 1.92.

Fig. 11 to 14 present the results for concrete of cement-water ratios of 1.58 and 2.25. Again we have a distinct improvement in resistance to disintegration with increase in cement content. For $c/w = 1.58$ the number of cycles at 25 per cent loss in weight varied from 248 for Cement 11 to 360 for Cement 9, and for $c/w = 2.25$ from 365 for Cement 13 to 510 for Cement 9 when standard portland cements are considered. The corresponding number of cycles varied from 306 to 328 and from 355 to 485 for the two mixes respectively, for concrete containing special portland cements. It is noted that except for the very lean mix of 1.25 cement-water ratio, the special portland cements did not offer higher resistance to freezing and thawing than did standard portland cements.

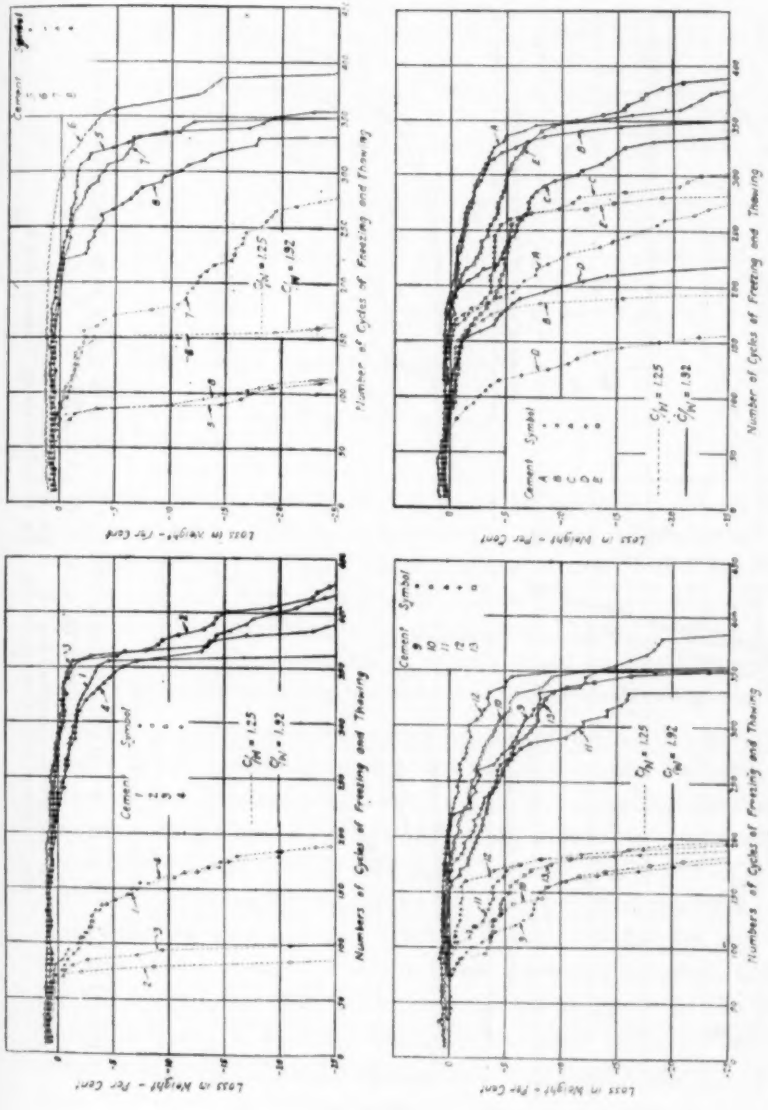


FIG. 7, 8, 9 AND 10—RATE OF DISINTEGRATION OF CONCRETE CONTAINING VARIOUS CEMENTS, C/W 1.25 AND 1.92

Fig. 7 (top left)—Cements 1 to 4
Fig. 8 (top right—Cements 5 to 8
Fig. 9 (bottom left)—Cements 9 to 13
Fig. 10 (bottom right)—Cements A to E

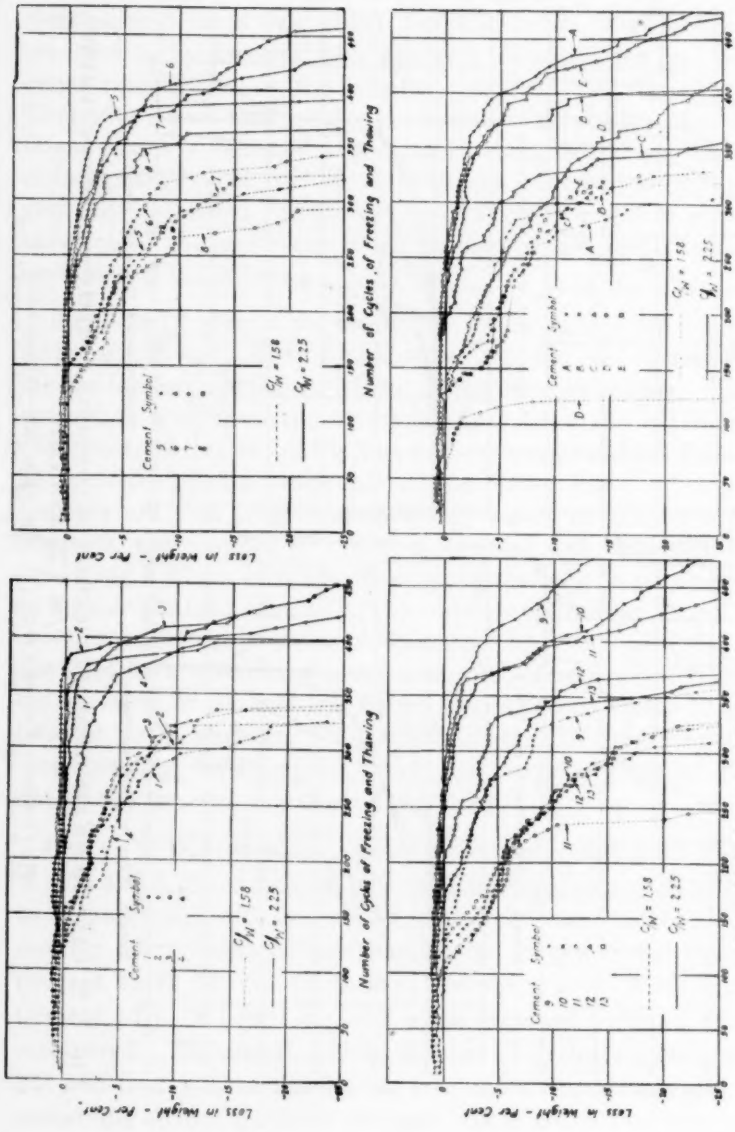


FIG. 11, 12, 13 AND 14—RATE OF DISINTEGRATION OF CONCRETE CONTAINING VARIOUS CEMENTS, C/W 1.58 AND 2.25

Fig. 11 (top left)—Cements 1 to 4
Fig. 12 (top right)—Cements 5 to 8
Fig. 13 (bottom left)—Cements 9 to 13
Fig. 14 (bottom right)—Cements A to E

TABLE 7—FREEZING AND THAWING RESULTS

Cement	Cycles at 25 Per Cent Loss of Weight Cement-Water Ratios of				
	1.25	1.58	1.92	2.25	Average
1	191	324	362	398	319
2	88	326	430	451	324
3	104	339	417	451	328
4	190	336	390	422	335
5	112	314	355	392	293
6	162	343	392	457	339
7	274	324	350	431	345
8	115	293	332	367	277
9	180	360	385	510	359
10	184	325	352	482	336
11	190	248	332	441	303
12	195	321	354	372	311
13	198	328	351	365	311
Average	160	322	369	426	320
A	274	306	380	485	361
B	193	323	350	405	318
C	305	318	335	355	328
D	156	125	219	414	229
E	282	328	389	470	367
Average*	264	320	364	429	344

*Cement D not included.

A study of the results in Fig. 7 to 14 inclusive, reveals that the number of cycles at five per cent loss in weight might as well be used as a criterion of durability as the number at twenty-five per cent loss; for already at five per cent the rate of loss in weight is so great that the actual number may readily be determined. It is therefore suggested that for practical consideration, results based on five per cent loss in weight be taken as the criterion of durability of the concrete.

In Table 7 the number of cycles which produced a 25 per cent loss in weight of the concrete is given for each cement and for each cement-water ratio. Average results are also given. Based on these averages two cements are found to have rather low durability. Cement B gave a durability index of only 277 and Cement 5 only 293 as compared with the average of 320 for all the standard portland cements. Cement 9 gave a durability index of 359 which is higher than for any other of the standard cements. Cement E gave the highest durability index for the special cements. Attention should be given to the fact that Cements 9 and E were produced by the same mill. Neither of these cements had special high strength values or other apparent differences from the other cements indicating that the process of manufacture may be of considerable significance.

Concrete containing Cement D was found to give indication of lower resistance to the action of freezing and thawing than obtained for the

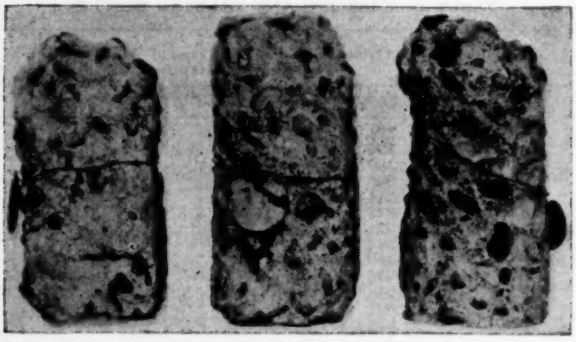


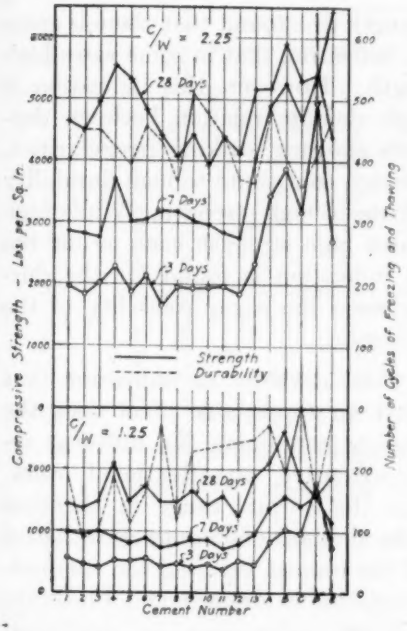
FIG. 15—TYPICAL CONCRETE SPECIMENS AFTER LOSING 25 PER CENT IN WEIGHT

standard and special portland cements. The durability index for this cement was only 229.

A typical sample of the appearance of the concrete specimens after they had lost about 25 per cent in weight is shown in Fig. 15. Often one or more of the three cylinders of each unit would split into two or more pieces before the test was concluded.

11. CORRELATION OF RESULTS

A study of the strength and durability results reveals that there is no apparent relation between these physical properties and the chemical composition and the chemical compounds of the different standard and high-early-strength portland cements. The average results in Table 6 indicate that Cement 12 is of relatively low strength and Cements 4 and 13 relatively high for standard cements. It is worth noting that Cements 4 and 13 were manufactured by the same mills which manufactured the special Cements B and C. None of these cements has either chemical composition or compounds materially different from the other brands. The higher strength values obtained with the special portland cements are probably due mostly to the finer grinding. As previously stated the gypsum content was also greater for the special than for the standard portland cements. Cement C had an unusual distribution of C_3S and C_2S, the C_3S being 62.6 per cent and the C_2S only 7.4 per cent. This, however, did not seem to add to the strength at the ages of test used in this investigation. The 200-mesh fineness of Cement C was much the same as for Cement B, but Cement B showed an average strength of 3550 p. s. i. as compared to 2970 for Cement C. The durability index of 318 for Cement B compared well with the index of 328 for Cement C.



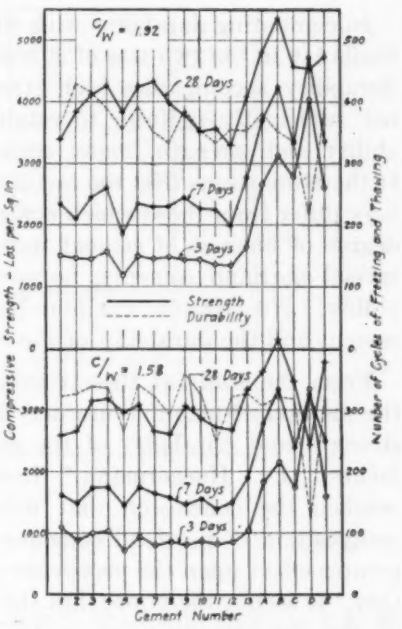


FIG. 16 (LEFT)—STRENGTH AND DURABILITY OF CONCRETES WITH C/W = 1.25 AND 2.25

FIG. 17 (RIGHT)—STRENGTH AND DURABILITY OF CONCRETES WITH C/W = 1.58 AND 1.92

In Fig. 16 and 17 both the strength and the durability are plotted for each cement and for each cement-water ratio, Fig. 16 containing results for $c/w = 1.25$ and 2.25, and Fig. 17 results for $c/w = 1.58$ and 1.92. The solid lines represent the strength and the broken lines the durability. The variation in strength for the different cements is readily noted and attention is called to the fact that in general when a cement is giving high strength at one age and one cement-water ratio it will also give high strength at all other ages and cement-water ratios.

Since all these cements came from the same district, the variation in concrete quality would be expected to be substantially less than for cements from different districts. In the report by Mr. Bates previously referred to, where thirty-two cements from various districts of the country were included, the results showed 3-day strengths varying from 600 to 1620 p. s. i., 7-day strengths from 1070 to 2780 p. s. i., and 28-day strengths from 2090 to 4030 p. s. i. for the 1:2.4:3.6 mix used, which is much greater variation than that found in this investigation.

In comparing durability with strength it is found that there is some similarity in the two sets of curves, indicating that in some cases high durability accompanies high strength. However, this indication is not sufficiently uniform to establish definite relation between durability and strength, many cements showing striking irregularities. It therefore seems that the factors which contribute to high durability may differ from those which contribute to high strength. While high degree of fineness of cement indicates high strength such as for the special portland cements, no such indication is found for the durability. No relation was found between the sugar solubility of the cement and the durability of the concrete.

From the study of these results there develops an indication that the method of manufacture may have an appreciable effect upon the strength and durability of the concrete containing the different cements. Dr. Haegermann,* from extensive investigational work, reached the conclusion that within the normal range in chemical composition of portland cements the method of manufacture had a greater effect upon the properties of the cement than did the composition. It seems, therefore, that the variation in strength and durability of the cements included in this investigation may be attributed primarily to the manufacturing process of each cement. A thorough study of the relation between the manufacturing process of the cement and the quality of the concrete should therefore be undertaken if we are to have cements of uniform qualities.

The effect of cement content on the durability of the concrete is brought out by Fig. 18. The maximum and minimum cycles for 25 per cent loss in weight are given for each cement-water ratio, thus presenting the total range of durability variation for all standard portland cements. This variation is so evident for all cement-water ratios that serious attention should be given to this particular problem on every important plain and reinforced concrete structure which is exposed to alternating freezing and thawing temperatures. Although considerable variation in the durability index is due to the testing process used, there is sufficient evidence that some cements had qualities well above the average. The selection of these outstanding commercial cements is a very important task for the concrete engineer.

The increase in cement content produced a distinctly beneficial effect upon the durability of the concrete regardless of brand. Both the upper and lower limits of the range in durability showed increased value with increased cement content. The average durability value

*"Zement-Tagungen in München," *Tonindustrie-Zeitung*, No. 73, Sept., 1934, p. 891.

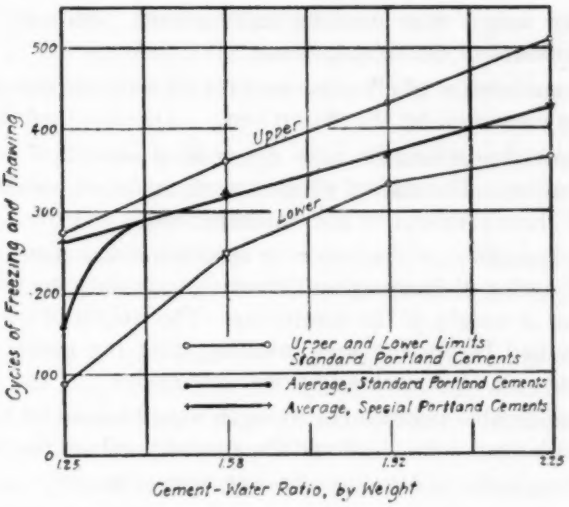


FIG. 18—EFFECT OF CEMENT CONTENT ON DURABILITY OF CONCRETE

for each cement-water ratio of the concrete has been plotted in Fig. 18 for all standard portland cements and also for all special portland cements. A straight line relationship was found to exist for all except the leanest mix containing standard portland cements, and for all the mixes containing special portland cements. The reason for the disproportionately low durability value for the concrete with cement-water ratio of 1.25 may be partially due to an important effect of the aggregate when this weak cement paste was used. For higher cement content per unit of water, the paste probably prevented the aggregate from playing an important role and the durability became a direct function of the quality of the cement paste. It has previously been shown that all cements gave approximately a straight line relation between strength and cement-water ratio. Both strength and durability thus follow the same law as far as the effect of the quality of the paste is concerned, both increase in value with increased cement concentration.

12. SUMMARY

1. The investigation reported herewith was carried out on 3 x 6-in. concrete cylinders made with eighteen different cements, thirteen of which were standard portland cements, four were special high-early-strength portland cements, and one a high alumina cement. Four different concrete mixes with cement-water ratios of 1.25, 1.58, 1.92

and 2.25 by weight were used for each cement. All mixes contained the same amount of net mixing water.

2. The consistency of all mixes and for all cements remained nearly constant as measured by the slump test.

3. Compressive strengths were determined after 3, 7 and 28 days of moist curing. Durability studies were made on specimens cured moist for 7 days previous to the exposure tests.

4. The durability of the concrete specimens was measured by the number of cycles of freezing and thawing necessary to produce a 25 per cent loss in weight of the specimens. The weight of each specimen was determined before the first freezing, after ten cycles of freezing and thawing and after each additional five cycles.

5. A considerable variation in strength was obtained for the different cements, both for the standard and the special portland cements.

6. The concrete specimens showed approximately straight line relations between strength and cement-water ratio for every cement and for each age of test, thus substantiating the strength formula: $S = A + B.\frac{c}{w}$, which develops into $S = A + K.c$ when w remains constant, that is: $\frac{B}{w} = K$.

7. The special portland cements gave higher strength of concrete than did the standard portland cements.

8. The high alumina cement gave concrete strength following approximately the straight line law, but showed no consistent increase in strength for length of moist curing in excess of three days.

9. The durability of concrete containing different cements showed considerable variation.

10. There was no consistent relationship between strength and durability of concrete containing different cements.

11. No definite relation could be found between strength and durability of the concrete and the chemical compounds for the different cements. The sugar solubility of the cement did not seem to have any relation to the durability of the concrete.

12. The durability of the concrete increased with the increase in the cement-water ratio of its paste. A straight line relationship was found for the average of all mixes containing special cements and of all but the very lean mix for standard cements.

13. For the very lean mix ($c/w = 1.25$) the special portland cements gave considerably higher durability than did the standard portland

cements. However, for intermediate and rich mixes there was no difference in the durability for the two types of portland cements.

14. The high alumina cement gave concrete of less durability than did the portland cements.

15. The variation in strength and durability of these different cements was probably more due to the method of manufacture of the cement than to any other cause.

16. It is recommended that the freezing and thawing test for the concrete be based on the number of cycles producing five per cent loss in weight.

17. Attention is called to the fact that considerable variations in both strength and durability are found both for standard portland cements and special high-early-strength portland cements, and that for important work these qualities should be ascertained before selecting the type and brand of cement to be used in the concrete.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by July 1, 1935.*

Water Retained in Hardened Cement Pastes*

BY RAYMOND WILSON† AND FRANK A. MARTIN‡

Many of the physical properties of concrete are dependent upon the quantity of water in the concrete. The strength is dependent upon the quantity of mixing water, upon the effectiveness of the methods of retaining that water during the early ages, and upon the quantity actually present when tested. Permeability to water and other liquids and resistance to many destructive agencies are dependent on the same factors. Volume changes are to a large extent dependent on changes in the water content of the cement paste.

It would appear that a knowledge of the influence of temperature and relative humidity on the equilibrium water content of hydrated cement pastes would be of assistance in interpreting phenomena associated with varying water content. There is little likelihood that such information will directly differentiate between free and combined water in a colloidal gel such as hydrated portland cement. There is, however, a possibility that such factors as influence the amount of water retained by the paste at a specific condition of temperature and humidity will have a parallel effect on the quantity of water actually forming an essential part of the paste structure.

It is the purpose of this paper to present data showing quantity of water remaining in cement pastes after drying, as influenced by the water-cement ratio, the duration of curing, and the temperature at which drying occurs. The tests were made in 1929-30. Since that time several articles have appeared in the literature describing tests on various aspects of the same subject.

MATERIALS AND METHODS OF TEST

Materials. Most of the tests were made using a single sample of cement, although other standard portland cements and a high early strength portland cement were used in some of the tests.

Preparation of Hydrated Cement. The cement and water were mixed to a uniform paste, and allowed to harden in tightly stoppered test tubes. The following quantities of mixing water were used: 30, 45, 60,

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75, and 90 per cent by weight of the cement. For the purpose of preventing segregation of the cement and water in the wetter mixtures, powdered feldspar was added. The quantities of the feldspar used were such that the mixtures were fluid enough to pour, but with little or no separation of water above the paste during the hardening period. Feldspar was chosen for this purpose because of its low ignition loss, and the slight probability of reaction with the calcium hydroxide set free during the process of hydration of cement.

One or two days after mixing, the hardened paste was removed from the tubes, ground to a powder passing a 28-mesh sieve, and stored in sealed containers in the laboratory. The grinding was done in the dense fog of the moist room in order to minimize loss of water during the period of grinding. Comparisons of duplicate specimens showed that the age at which the paste was ground, whether after 24 hr. of hardening, or just before test, had no detectable effect on the results. For convenience the hardened mixtures were ground between 24 and 48 hr. after making.

Determination of Water Retained after Drying. The powdered pastes were dried in porcelain crucibles in a specially constructed container which was set in an electrically heated and controlled constant temperature oven. The container was ventilated by a slow stream of dry carbon-dioxide-free air brought to the oven temperature by passing it through a coil attached to the container. The general procedure was to continue the heating for 24 hr. As can be seen from the data in Table 1, heating for longer periods than 24 hr. brought about no further loss in weight even at 50°C., the lowest drying temperature used. This method was used for drying at temperatures up to 200°C.

For the few tests made at higher temperatures, the crucibles were covered and placed in an electric muffle furnace which had previously

TABLE 1—EFFECT OF DURATION OF DRYING ON WATER RETAINED

Drying Temp.	50°C.				115°C.				200°C.			
Curing Period	3d.		14d.		3d.		14d.		3d.		14d.	
W/C by Weight	44.5	75.8	45.3	76.0	44.2	72.5	45.3	76.6	45.8	75.3	44.7	78.2
Duration of Drying Hours	Water Retained—% of Dry Cement											
2	34.0	56.8	39.1	60.3	8.9	10.5	13.0	15.1	5.7	6.8	8.7	10.3
4	24.4	34.3	33.0	43.6	8.2	10.3	12.1	14.3	5.7	6.9	8.7	10.3
7	15.9	18.9	23.1	25.7	7.6	9.2	11.3	13.2	5.4	6.6	8.4	9.8
17	11.2	13.6	15.1	18.0	——	——	——	——	——	——	——	——
24	10.8	13.3	15.1	18.0	7.5	9.3	11.3	13.4	5.5	6.9	8.5	10.0
48	10.4	13.2	15.1	18.2	7.6	9.3	11.3	13.4	5.4	6.7	8.4	10.0
72	10.4	13.0	15.5	18.4	7.5	9.2	11.3	13.3	——	——	——	——
96	10.6	13.1	15.9	18.6	——	——	——	——	——	——	——	——

been adjusted to the desired temperature. There was no means of controlling the temperature closely and a somewhat increased opportunity was presented for contamination of the paste by carbon dioxide in handling and heating. These tests at temperatures above 200°C. were made on samples previously dried at one of the lower temperatures. Heating at the higher temperatures was continued for 1 hr., which was apparently sufficient to bring the paste to constant weight.

After being weighed, the sample, dried by one of the methods described, was ignited at about 1000°C. and again weighed. This final loss in weight, corrected for the loss on ignition of the cement, was taken as the water retained in the paste after drying, while the total loss, similarly corrected, was taken as the total water content of the paste when the test was started. Precautions taken in mixing and handling the pastes insured the essential accuracy of these assumptions, which was also confirmed by occasional analyses of pastes.

DISCUSSION OF TEST DATA

The effect of quantity of mixing water, curing period, and drying temperature for certain representative ages are shown graphically in Fig. 1. These data bring out the inter-relations of the amount of water retained to the water-cement ratio, the curing period, and the drying temperature.

Effect of Quantity of Mixing Water. It is seen from Fig. 1 that the quantity of mixing water used has a noticeable effect on the quantity of water remaining in the pastes after drying, which is a general characteristic through the whole range of variables included in these tests. The curves in these figures have been projected to the origin. In the region of mixing water contents above about 40 per cent there appears to be an approximately linear relationship between increase in mixing water and increase in water retained after drying. The more rapid decrease in water retained with decreasing mixing water as indicated by the left-hand portion of these curves may indicate that at low water-cement ratios insufficient water is present to satisfy the reactions which take place at high water ratios. This speculative observation is reinforced by the fact that at early ages it applies only to the 30 per cent pastes while at later ages it also applies to the 45 per cent pastes.

If it may be assumed that there is a parallel between the results of these tests and the degree or extent of hydration (using the term loosely), it would appear that hydration is more complete or farther advanced in mixtures of high water-cement ratios than in those of low water content. In a general way, strength increases with more complete hydration of the cement. It appears, therefore, that the lowered

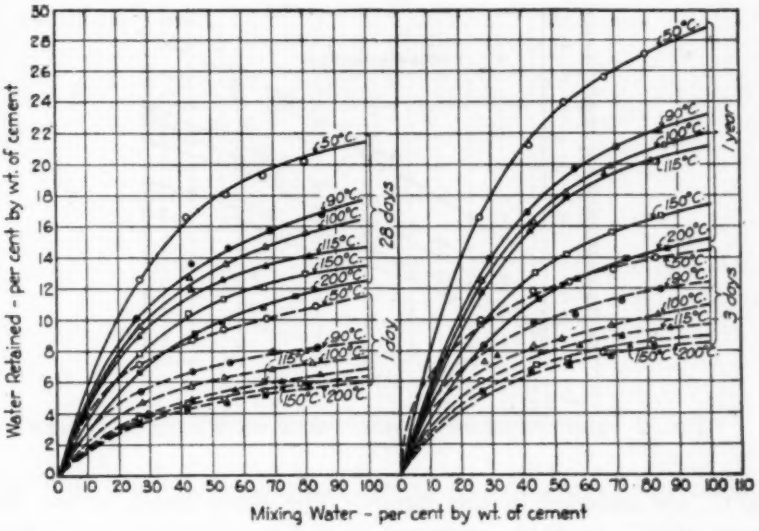

FIG. 1—EFFECT OF WATER-CEMENT RATIO ON WATER RETAINED

strengths resulting from excessive water are almost entirely due to the physical effect of greater dispersion, which masks any slight effect due to differences in hydration within the range of water contents used in concrete mixtures.

Effect of Duration of Curing Period. In Fig. 2, which shows the effect of curing period on the retention of water, the age has been plotted to a logarithmic scale for convenience. The relationship plotted is that for a mixture of cement with 60 per cent mixing water, the values being taken from curves in Fig. 1 and similar curves for other ages. The curve showing the relationship between age and water retained is the same type of curve as that found for the relationship between age and strength for similar curing conditions.

Effect of Drying Temperature. In Fig. 3, water retained after drying is plotted against drying temperature. Here, too, the data for the 60 per cent water paste only are given, though similar relationships were found in pastes of other mixing water contents. The values are taken from the curves of Fig. 1 and other similar curves not given in this paper. At ages of 28 days, or greater, the curves show a gradual decrease in the amount of water retained as the drying temperature rises. At early ages, there is an indication of a break at about the temperature of boiling water. The few tests made at temperatures above

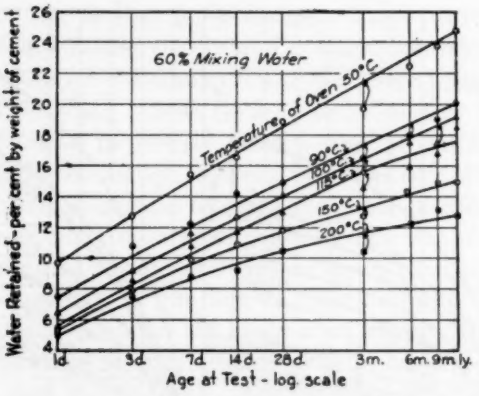

FIG. 2—EFFECT OF AGE WHEN DRIED ON WATER RETAINED

200°C. are included in this figure. The sharp break in the curve above 400°C. probably corresponds to the dehydration of calcium hydroxide. Dehydration is apparently not complete at 600°C., though the methods of test employed for the high temperatures do not insure the absence of carbonation. Any carbon dioxide absorbed during drying would appear as fixed water.

Effect of Composition and Fineness of Cement. These tests were repeated in part on several portland cements including a high early strength cement. The factors studied had similar effects on all cements, though there were differences among them. The data were insufficient to justify any attempt at correlating the quantity of water retained with composition and fineness. It may be noted that the high early strength cement retained substantially more water than the standard cements, for comparable test conditions.

Effect of Temperature of Curing. The rate of hardening of cement mixtures is known to be affected by temperature and it may be expected that differences would be found in the quantity of water retained by cement which had hydrated and cured at various temperatures. Comparative tests were made on 4 mixtures as follows:

(1) Mixed at 5°C., kept at 5°C. until test.

(2) Mixed and cured 2 days at 5°C., thereafter cured at room temperature.

(3) Mixed at 21°C., cured at 21°C. until test.

(4) Mixed and cured 2 days at 40°C., cured thereafter at 21°C.

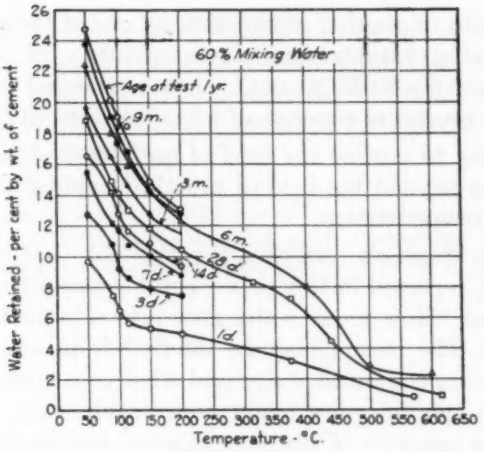

FIG. 3—EFFECT OF DRYING TEMPERATURE ON WATER RETAINED

The materials were brought to the desired temperatures, mixed quickly, and returned to the temperature bath. An ice bath which maintained a temperature close to 5°C. and an electrically controlled oven at 40°C. were used as baths. Results interpolated for a 60 per cent water mixture are shown in Fig. 4. Larger quantities of water are retained as the temperature of initial curing is increased, though the differences are rather smaller than might have been expected.

General Discussion. The data presented in this report find some more or less practical applications and contribute toward the better understanding of some properties of cement mixtures. With their help it is apparently possible to design concrete mixtures to meet a specification for absorption or to calculate the absorption of a mixture of known proportions and curing with accuracy approximating the reproducibility of absorption tests. This question has received more ex-

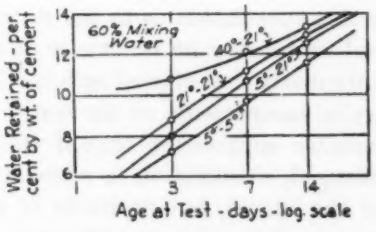

FIG. 4—EFFECT OF CURING TEMPERATURE AND AGE ON WATER RETAINED

tended treatment in another publication by one of the authors.* The differences among various cements in quantity of water retained become of relatively little moment when considered in terms of absorption of the mortar or concrete of which the cement becomes a part.

In attempting to outline the field of usefulness of these data, it is necessary to go beyond the field of practical application and venture into the realm of speculation.

The authors expressly disclaim the thought that the values for water retained, reported in this paper, can be considered as an answer to the question: "How much water goes into combination with portland cement?" The reason may be inferred from another question: "What constitutes combined water and what constitutes free water?" The answer to this question might conceivably vary, depending on what particular property of the cement paste was under scrutiny. If disruption by expansion of freezing water were being studied, the answer might be: "Free water is water that is liquid and can freeze; combined water is in a state of association such that it already functions as a solid." If the question is one involving permeability of cement paste to the flow of water the answer would appear to be "Free water is mobile water; combined water is that which is fixed in position." The quantity of combined water by these two criteria may be the same, but, alike or different, it is probably higher than the amount left in the paste after drying under any of the conditions of these tests.

It is believed, however, that the various factors studied would influence the amount of water in true combination in the same manner as they influence the amount of water left in the paste dried as in these tests. For identical conditions of age and temperature, the amount of combined water in a paste mixed with a high water content would be greater than in one mixed with a low water content. Also the quantity of combined water would increase as moist curing was prolonged.

The relation between drying temperature and water retained affords a basis for an interesting speculation as to the state of combination of water in concrete and its bearing on resistance of concrete to certain disruptive effects, particularly that of frost action. The curves in Fig. 3 show the effect of drying temperature on the water retained in a dried paste. It is a reasonable supposition that if the temperature of a saturated paste is changed, dissociation or association will occur in a manner indicated by the curves, but probably at consistently higher levels. If so, heating a paste would convert some combined water to

*"Limitations of the Absorption Test for Concrete Products," *Proceedings*, American Concrete Institute, 1929, V. 25, p. 552.

free water, and cooling a paste containing free water would result in some of the free water being taken into combination. If so, a paste which is slowly cooled and frozen may contain relatively little water which can be converted to ice on freezing, with a correspondingly small internal expansion.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by July 1, 1935.*

Mass Concrete Tests in Large Cylinders*

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INTRODUCTION

THE PROGRAM of mass concrete research conducted by the Bureau of Reclamation in the Denver laboratories in connection with the design and construction of Boulder dam included tests in cylinders varying in size from 2 x 4 to 36 x 72 in. to determine the effect of various factors on compressive strength, Young's Modulus and Poisson's Ratio. The concretes used in these tests included mixes in which the maximum size of aggregate varied from ¾ to 9 in. and from full mass concrete with 9-in. aggregate to wet-screened concrete having ⅜-in. maximum particles.[1]

The data obtained from the Boulder dam investigations have evaluated, for concretes with Arizona aggregates, the following variable factors in terms of indicated compressive strength, elastic properties and physical characteristics:

1. Size of test cylinder
2. Maximum size of aggregate
3. Wet-screening

In addition, the relationship between full mass concrete tested in large cylinders and wet-screened concrete tested in small cylinders, as is customary in field control tests, has been established, together with a comparison between standard (fog) and mass curing for various placing temperatures.[2,3]

MATERIALS AND METHODS OF TEST

The aggregates used in the tests were obtained from the Arizona gravel deposit, the source of supply for Boulder dam, and have been

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[1]References will be found at the end of the paper.

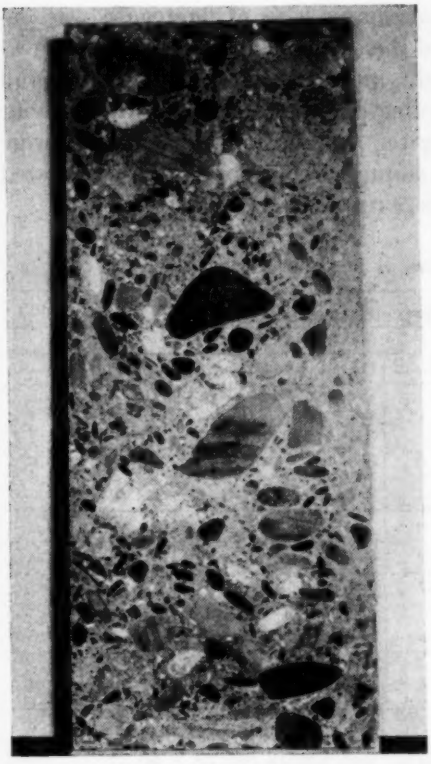


FIG. 1—2 BY 5-FT. VERTICAL CROSS SECTION OF BOULDER DAM MASS CONCRETE

described in earlier publications.[2] The physical characteristics of these materials are given in Table 1.

The cement used was a standard portland cement except that a special heat treatment was applied to the clinker of a relatively small lot, manufactured in a commercial plant, to reduce the heat of hydration of the finished cement. The physical and chemical characteristics of the cement are presented in Table 2.

The mass concrete investigations included two main series of tests as outlined in Tables 3 and 4. Series I (Table 3) is based upon three basic mass concrete mixes, all having 9-in. maximum size aggregate, a constant aggregate grading, and containing 0.80, 1.00 and 1.20 bbl. of cement per cu. yd. of concrete respectively. The water-cement ratios for the three mixes were varied, as indicated in the outline, to maintain a constant slump of approximately 3 in. With each basic mix, companion mixes were cast in which the water-cement ratio, cement-sand ratio, and the slump were maintained the same as for the

corresponding basic mix. The maximum size of aggregate was decreased to 6, 3, 1½, and ¾ in. and the amount of coarse aggregate was decreased and the cement content increased accordingly. The full mixes were tested in cylinders having diameters equal to at least four times the maximum size of aggregate. In addition, the concretes were wet-screened to 3 and 1½ in. maximum, where applicable, and tested in 8 x 16-in. and 6 x 12-in. cylinders respectively.

TABLE 1—PHYSICAL PROPERTIES OF BOULDER DAM AGGREGATES

Test	Individual % Retained (Used in Both Series)							
	Sand		Gravel					
	Sieve	No. 4 Max.	Screen	¾″ Max.	1½″ Max.	3″ Max.	6″ Max.	9″ Max.
Sieve Analysis	No. 4	0	9″					0
	No. 8	17.0	6″				0	9.4
	No. 14	11.6	3″			0	23.9	21.6
	No. 28	15.0	1½″		0	34.0	25.9	23.5
	No. 48	41.6	¾″	0	45.5	30.0	22.8	20.6
	No. 100	12.4	⅜″	58.9	32.1	21.2	16.1	14.6
	Pan	2.4	No. 4	41.1	22.4	14.8	11.3	10.3
	Total	100.0	Total	100.0	100.0	100.0	100.0	100.0
Fineness Modulus		2.72		6.59	7.23	7.83	8.25	8.59
Specific Gravity		2.64		2.67	2.69	2.72	2.70	2.69
Lb./cu. ft., dry rodded		109.6		103.8	109.6	115.8	119.5	121.0
% Voids, dry rodded		33.4		37.7	34.7	31.8	29.1	27.9

TABLE 2—PHYSICAL AND CHEMICAL PROPERTIES OF THE CEMENT

Chemical Analysis		Compound Composition		Physical Properties	
				Passing 200 Mesh Sieve	90.1%
SiO_2	23.09%	C_3S	52.9%	Specific Surface	2025 sq. cm. per g.
Al_2O_3	4.64	C_2S	26.3	Initial Time of Set	2 hr. 30 min.
Fe_2O_3	1.95	C_3A	9.0	Final Time of Set	4 hr. 45 min.
CaO	65.54	C_4AF	5.9	Specific Gravity	3.11
SO_3	1.38	$CaSO_4$	2.3	Normal Consistency	20.8%
MgO	1.60			A.S.T.M. Standard Briquets:	
Free Lime	0.08	Free Lime	0.1	Avg. Tensile Str. 7 da.	270 lb. per sq. in.
Loss on Ignition	1.38	Loss on Ignition	1.4	Avg. Tensile Str., 28 da.	365 lb. per sq. in.

The basic mix in Series II (Table 4) was the same as the basic mass-concrete mix containing 1.00 bbl. of cement per cu. yd. of concrete used in Series I. It should also be mentioned that this mix is practically identical as to cement content and aggregate grading with the field mass-concrete mix being used in Boulder dam. The companion mixes for this series were obtained by simply omitting successive sizes of the coarse aggregate with very slight adjustments in the remaining materials to make the mixes conform exactly to what would have been obtained by actually wet-screening the full mass mix to varying maxi-

TABLE 3—OUTLINE OF SERIES I

SERIES I. 3 in. slump for all mixes.
Sand grading the same for all mixes.
Gravel grading in each mix is the same as that in the 9 in. max. size mix, but with the larger sizes omitted.
G/S ratio of mixes A1, B1, C1 are the same. Similarly for mixes A2, B2, C2, etc.

A. Basic mix, 0.8 bbl. cem. per cu. yd. with 9 in. max. size aggregate. w/c — 0.66 for all mixes. Cement to sand = 1:3.06 for all mixes.

1.	9	in. max. size agg.,	8.81	pts. of coarse agg.,	2.88	= G/S,	0.82	bbl. cem. per cu. yd.,	tested in 36 by 72 in. cyl.,						wet-screened to 3 in. and 1½ in. max. sizes and tested in 8 and 6 in. diam. cyl.								
2.	6	" " ,	8.08	" " ,	2.64	" ,	0.87	" " ,	" "	24	by	48	" " ,		"	"	"	"	"	"			
3.	3	" " ,	6.62	" " ,	2.16	" ,	0.97	" " ,	" "	12	by	24	" " ,		"	"	"	"	"	"			
4.	1½	" " ,	5.43	" " ,	1.77	" ,	1.09	" " ,	" "	6	by	12	" "										
5.	¾	" " ,	4.50	" " ,	1.17	" ,	1.17	" " ,	" "	6	by	12	" "										

B. Basic mix, 1.0 bbl. cem. per cu. yd. with 9 in. max. size aggregate. w/c = 0.54 for all mixes. Cement to sand = 1:2.45 for all mixes.

1.*	9	in. max. size agg.,	7.05	pts. of coarse agg.,	2.88	= G/S,	1.02	bbl. cem. per cu. yd.,	tested in 36 by 72 in. cyl.,						wet-screened to 3 in. and 1½ in. max. sizes and tested in 8 and 6 in. diam. cyl.								
2.	6	" " ,	6.47	" " ,	2.64	" ,	1.07	" " ,	" "	24	by	48	" " ,		"	"	"	"	"	"			
3.	3	" " ,	5.30	" " ,	2.16	" ,	1.19	" " ,	" "	12	by	24	" " ,		"	"	"	"	"	"			
4.	1½	" " ,	4.35	" " ,	1.77	" ,	1.33	" " ,	" "	6	by	12	" "										
5.	¾	" " ,	3.60	" " ,	1.47	" ,	1.44	" " ,	" "	6	by	12	" "										

C. Basic mix, 1.2 bbl. cem. per cu. yd. with 9 in. max. size aggregate. w/c = 0.47 for all mixes. Cement to sand = 1:2.04 for all mixes.

1.	9	in. max. size agg.,	5.88	pts. of coarse agg.,	2.88	= G/S,	1.20	bbl. cem. per cu. yd.	tested in 36 by 72 in. cyl.,						wet-screened to 3 in. and 1½ in. max. sizes and tested in 8 and 6 in. diam. cyl.								
2.	6	" " ,	5.39	" " ,	2.64	" ,	1.25	" " ,	" "	24	by	48	" " ,		"	"	"	"	"	"			
3.	3	" " ,	4.41	" " ,	2.16	" ,	1.40	" " ,	" "	12	by	24	" " ,		"	"	"	"	"	"			
4.	1½	" " ,	3.62	" " ,	1.77	" ,	1.54	" " ,	" "	6	by	12	" "										
5.	¾	" " ,	3.00	" " ,	1.47	" ,	1.66	" " ,	" "	6	by	12	" "										

*The same as mix B1 in Series II, and very nearly the same as the mas mix used in Boulder dam.

TABLE 4—OUTLINE OF SERIES II

SERIES II. All mixes obtained by wet-screening mix B1.
The sand grading is the same in all mixes.
Gravel grading in each mix is the same as that in mix B1, but with the larger sizes omitted.

B. Basic mix, 1.0 bbl. cem. per cu. yd., with 9 in. max. size agg., and 3 in. slump.
Mix proportions determined by an analysis of the material removed by wet-screening.
3-in. slump for all mixes, when mixed with the cobbles and then wet-screened.

1.*	9 in. max. size agg.,	w/c = 0.54,	Mix = 1:2.45:7.05,	2.88 = G/8,	1.02 bbl. cem. per cu. yd.,	Tested in 18, 24, and 36 in. diam. cyls
6.	Mix B1	wet-scr. to 6	in. max. size agg.,	3	in. slump when mixed without 6-9 in. cobbles,	w/c = 0.54	Mix = 1:2.45:6.41,	2.62 = G/8,	1.08 bbl. cem. per cu. yd.	Tested in 18, 24, and 36 in. diam. cyl.
7.	"	"	"	3	"	"	3¾ in. slump when mixed, without 3-9 in. cobbles	"	0.54,	"	1:2.45:4.93,	2.01	"	1.23	"	"	Tested in 6, 8, 12, 18, 24, and 36 in. diam. cyl.
8.	"	"	"	1½	"	"	5	in. slump when mixed without 1½-9 in. cobbles	"	0.53,	"	1:2.44:3.30,	1.35	"	1.49	"	"	Tested in 3, 6, 8, 12, 18, 24, and 36 in. diam. cyl.
9	"	"	"	¾	"	"	8¼ in. slump when mixed, without ¾-9 in. cobbles	"	0.55,	"	1:2.51:1.88,	0.75	"	1.78	"	"	Tested in 2, 3, 6, 8, 12, 18, and 24 in. diam. cyl.
10.	"	"	"	⅜	"	"	9	in. slump when mixed, without ⅜-9 in. cobbles	"	0.54,	"	1:2.52:0.75,	0.30	"	2.14	"	"	Tested in 2, 3, 6, 8, 12, and 18 in. diam. cyl.

*The same as mix B1 in Series I, and very nearly the same as the mass mix used in Boulder dam.

mum sizes down to ⅜ in. These slight adjustments represent materials, principally fine sand and a little cement, that would have been lost in the wet-screening operation.

As will be seen by the data presented, the procedure of mixing the concretes without the larger sizes of aggregates resulted in a gradually increasing slump as the maximum size of aggregate was decreased; whereas actually wet-screening to successively smaller sizes had no noticeable effect on the consistency of the resulting concretes. The full mass mix and each of the equivalent wet-screened mixes were tested in various sizes of cylinders within reasonable minimum limits of the ratio of diameter of cylinder to maximum size of aggregate, as indicated in the outline (Table 4).

The methods of mixing, casting, testing, etc., were adopted only after numerous preliminary investigations had been made to insure satisfactory and uniform results. The adopted procedure largely eliminated differences that might have been caused by these operations.[4]

All materials, including each separated size of aggregate, were weighed separately for each batch, and the concrete was machine mixed for five minutes. The concrete was compacted in the test cylinder molds with an internal vibrator, and the specimens were stored in fog curing rooms maintained at 70°F. Specimens were prepared for testing by grinding the top surfaces. Frame type compressometers and extensometers were used for measuring the elastic properties of the concrete, and compressive strength tests were made in a hydraulic type testing machine, which applied the load at a uniform rate of 17 pounds per square inch per second.[5]

DISCUSSION OF DATA

General—Average mix data, concrete properties, and strength and elastic properties for all tests are presented in Tables 5 and 6. Measures of accuracy indicate that the tabulated values are reliable. Preliminary tests proved the efficiency of the adopted procedures, and mean variations of individual results from the average indicate a high degree of accuracy for the final results. Mean variations of individual values from the average were well under five per cent for most groups of specimens.

Effect of size of test specimen—The fact that size of test cylinder exerts a marked influence upon the indicated strength of concrete has long been a matter of common knowledge. Table 6 provides data for evaluating the effect of varying size of cylinder on the strength and

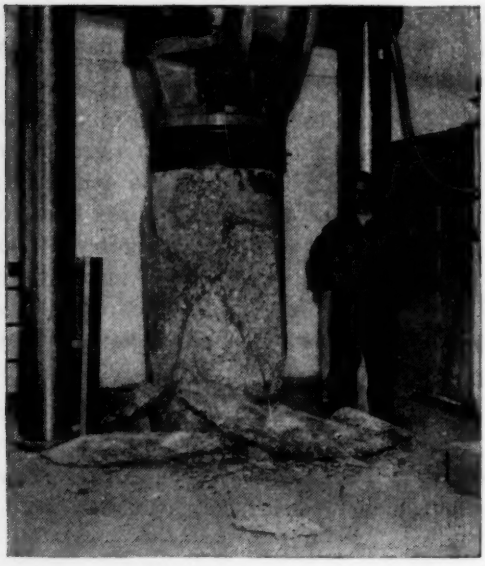


FIG. 2—A 36 BY 72-INCH CYLINDER AFTER FAILURE

elastic properties of concretes composed of Arizona aggregates. When these data were tabulated according to size of cylinder, including for any given size all tests in which the diameter of specimen was equal to or greater than four times the maximum size of aggregate, it was found that the strengths for any given size of cylinder were practically equal regardless of the size of aggregate. Hence, all tests for a given size of cylinder were averaged to obtain strength values for the different sizes of specimens. The individual and average results of such an analysis are shown graphically in Fig. 3 and 4. Since age apparently has no effect on the cylinder size-strength relationship the 7-day values are not included in these diagrams because the tests at this age did not cover the full range of cylinder sizes. Fig. 4 shows that the average results for effect of size of specimen forms a smooth curve; and that with the standard 6 x 12-in. cylinder as the basis of comparison at 100 per cent strength, that the same concrete will yield 108 and 84 per cent strengths when tested in 2 x 4 and 36 x 72-in. cylinders respectively. It should be borne in mind that these values apply for conditions in which the ratio of diameter of cylinder to maximum size of aggregate is equal to or greater than four.

A complete explanation of the reduction in the strength of concrete due to increasing the size of specimen has not as yet been satisfactorily

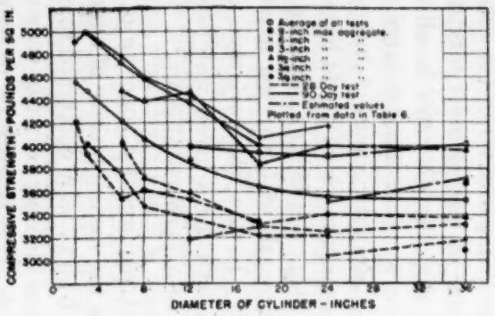
developed. It is probable that the ratio of diameter of cylinder to maximum size of aggregate, internal stresses, end restraint and other factors not yet revealed combine to produce the final result. Additional tests are at present in progress in the Bureau of Reclamation laboratories which are designed to further explain the cylinder size variable.

Apparently, variations in the size of test cylinder have no effect on Young's modulus or Poisson's ratio as may be seen from a study of Table 6 and Fig. 5.

Limiting value for ratio of diameter of cylinder to maximum size of aggregate—Past experience has indicated that it is undesirable to test concrete under conditions such that the diameter of test cylinder is less than four times the maximum size of aggregate. A study of the data in Table 6 indicates a decided and consistent drop in strength when the size of aggregate exceeds one fourth of the cylinder diameter for maximum aggregates of 1½ inches or less. For maximum sizes of aggregate of 3 in. or more there is a reduction when the ratio is equal to or slightly less than one to four. Also, there is a slight trend toward an increase in the value of mean variations as the ratio of diameter to maximum size is decreased. However, the individual results obtained with large cylinders are much more consistent than those obtained with smaller specimens.

Effect of maximum size of aggregate—The data presented in Table 5 show the effect of varying maximum size of aggregate on the properties of concretes when the proportions of coarse aggregate are varied along with the maximum size to maintain the slump and water-cement ratio constant. It will be noted that the size of test cylinder used in the series under discussion was varied so that the diameter in all cases, except for the ¾-in. aggregate concretes, was four times as large as the maximum size aggregate. When the strength values tabulated in Table 5 are reduced to the equivalent strengths which would have been obtained in 36 x 72-in. cylinders (by using the factors obtained from Table 6) no consistent variation in strength with varying maximum aggregate is found, as shown by Fig. 6. In fact, the variations of the different mixes from the average strength for a given water-cement ratio are in all cases within the limits of experimental error.

Fig. 7 shows the increase in cement required for concrete of fixed consistency and water-cement ratio, as the maximum size of aggregate is decreased. The data shown in Figs. 6 and 7 were combined to obtain the cement efficiency (pounds per square inch per barrel of cement per cubic yard of concrete) curves of Fig. 8. It is interesting to note that about the same strength per unit of cement is obtained for a given


COMPRESSIVE STRENGTH - POUNDS PER SQ IN.
DIAMETER OF CYLINDER - INCHES
Average of all tests
6-inch max aggregate
28 Day test
90 Day test
Estimated values
Plotted from data in Table 6
PER CENT OF STRENGTH IN 6"X12" CYLINDER
Average of all values shown in Fig. 3
DIAMETER OF CYLINDER - INCHES

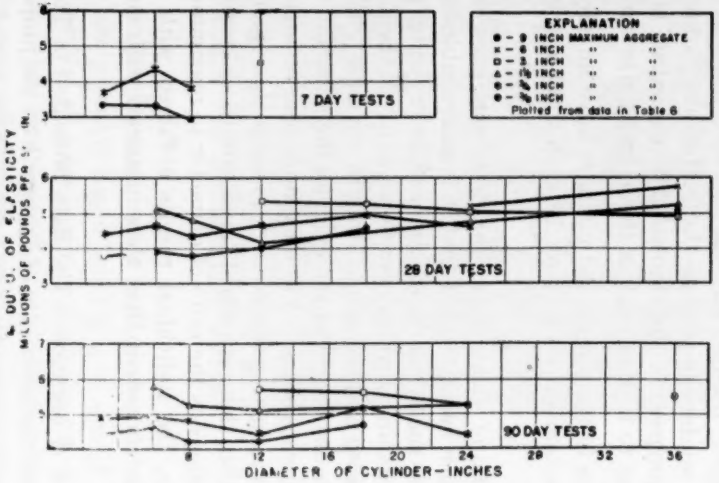


FIG. 3 (TOP LEFT)—CONCRETE STRENGTH IN CYLINDERS OF VARIOUS SIZES

FIG. 4 (BOTTOM LEFT)—EFFECT OF SIZE OF TEST CYLINDER ON THE COMPRESSIVE STRENGTH OF CONCRETE

FIG. 5 (ABOVE)—EFFECT OF SIZE OF TEST CYLINDER ON THE MODULUS OF ELASTICITY OF CONCRETE

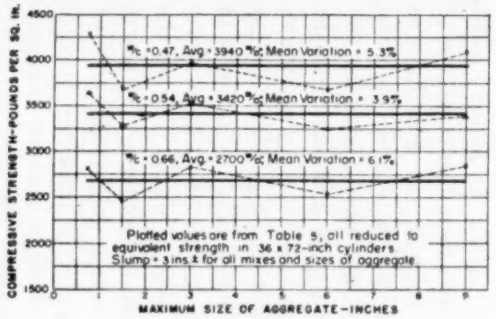
TABLE 5—CONCRETE PROPERTIES AND TEST DATA FROM SERIES I

TABLE 5—CONCRETE PROPERTIES AND TEST DATA FROM SERIES I

No.	Mix Data: Parts by Weight C:S:G	W/C	Max. Size Agg.-In.	Concrete Properties: Slump-In.	U. Wt. Lb. per cu. ft.	Abs. Vol. Paste Per Unit Vol. of Concrete	Cement Cont. Bbl. per cu. yd.	Water Content-Lb. per cu. yd.	Cylinder Size In.	Strength and Elastic Properties: 7 Day Tests U. Str.-Lb. per sq.in.	7 Day Tests Y. M. x 10^{-6}	7 Day Tests P. R.	28 Day Tests U. Str. Lb. per sq. in.	28 Day Tests Y. M. x 10^{-6}	28 Day Tests P. R.	90 Day Tests U. Str.-Lb. per sq.in.	90 Day Tests Y. M. x 10^{-6}	90 Day Tests P. R.	1 Year Tests U. Str. Lb. per sq.in.	1 Year Tests Y. M. x 10^{-6}	1 Year Tests P.R.
A-1	1:3.06:8.81	0.66	9	3	155.0	0.18	0.82	203	36x72				2600	4.67	0.17	3120	5.40	0.18	3570	6.10	0.18
	Wet-screened	0.66	3	3	153.3	0.23	1.03	256	8x16	1890	4.30	0.17	2670	5.18	0.19	3500	5.60	0.19	4110	6.10	0.20
	Wet-screened	0.66	1½	3	151.7	0.27	1.25	310	6x12	2190	4.20	0.20	3290	5.04	0.21	4120	5.40	0.21	4840	6.00	0.21
A-2	1:3.06:8.08	0.66	6	2¾	154.4	0.19	0.87	216	24x48				2290	4.50	0.17	2890	4.90	0.19	3700	5.60	0.18
	Wet-screened	0.66	3	2¾	153.6	0.22	1.02	253	8x16	1850	4.18	0.18	2720	5.10	0.17	3380	5.20	0.17	4110	6.20	0.19
	Wet-screened	0.66	1½	2¾	152.1	0.27	1.24	308	6x12	2080	4.10	0.20	3120	5.15	0.21	3980	5.50	0.22	4800	6.00	0.22
A-3	1:3.06:6.62	0.66	3	3½	152.7	0.21	0.97	241	12x24				2560	4.10	0.15	3130	5.40	0.19	3720	6.20	0.25
	1:3.06:6.62	0.66	3	3½	153.0	0.21	0.98	243	8x16	1910	4.40	0.17	2510	4.80	0.18	3280	5.50	0.18	4050	5.90	0.20
	Wet-screened	0.66	1½	3½	151.5	0.26	1.20	298	6x12	2060	4.25	0.20	2790	4.90	0.19	3490	5.60	0.23	4160	6.30	0.20
A-4	1:3.06:5.43	0.66	1½	2	153.8	0.24	1.09	270	6x12	1830	4.10	0.19	2570	5.00	0.20	3150	5.30	0.20	3720	6.20	0.23
A-5	1:3.06:4.50	0.66	¾	2	150.3	0.26	1.17	290	6x12	2050	3.90	0.19	2920	4.80	0.22	3650	5.20	0.21	4510	5.95	0.21
B-1	1:2.45:7.05	0.54	9	3	155.6	0.19	1.02	205	36x72				3090	5.10	0.18	3680	5.50	0.20			
	Wet-screened	0.54	3	3	153.3	0.24	1.23	250	8x16	2420	4.90	0.18	3470	5.40	0.19	4220	6.00	0.19	4960	6.60	0.19
	Wet-screened	0.54	1½	3	150.5	0.28	1.49	297	6x12	2880	4.70	0.20	4050	5.40	0.21	4810	5.90	0.21	5970	6.40	0.22
B-2	1:2.45:6.47	0.54	6	3¼	155.9	0.21	1.07	217	24x48				3020	5.20	0.18	3540	5.60	0.18	4110	6.20	0.18
	Wet-screened	0.54	3	3¼	154.4	0.24	1.26	256	8x16	2420	4.90	0.18	3470	5.40	0.19	4220	6.00	0.19	4960	6.60	0.19
	Wet-screened	0.54	1½	3¼	151.0	0.29	1.50	305	6x12	2880	4.70	0.20	4050	5.40	0.21	5180	5.90	0.21	5970	6.40	0.22
B-3	1:2.45:5.30	0.54	3	3¼	154.1	0.23	1.19	242	12x24				3180	4.60	0.14	3880	5.50	0.17	4210	6.40	0.22
	1:2.45:5.30	0.54	3	3¼	153.7	0.23	1.20	244	8x16	2410	4.90	0.18	3290	5.15	0.17	3830	5.80	0.18	4910	6.80	0.21
	Wet-screened	0.54	1½	3¼	152.2	0.28	1.46	296	6x12	2740	4.80	0.20	3520	5.40	0.23	4110	5.90	0.20	4980	6.60	0.21
B-4	1:2.45:4.35	0.54	1½	3	154.8	0.26	1.33	270	6x12	2600	4.90	0.20	3560	5.50	0.20	4060	6.30	0.22	4990	6.70	0.22
B-5	1:2.45:3.60	0.54	¾	3¼	152.3	0.28	1.44	292	6x12	2810	4.60	0.20	3740	5.30	0.22	4730	5.80	0.23	5670	6.50	0.22
C-1	1:2.04:5.88	0.47	9	3½	156.4	0.21	1.20	212	36x72				3740	5.50	0.18						
	Wet-screened	0.47	3	3½	154.6	0.26	1.48	262	8x16	2980	5.10	0.20	3790	5.66	0.19	4640	5.60	0.16	5200	6.90	0.19
	Wet-screened	0.47	1½	3½	153.2	0.31	1.78	315	6x12	3030	4.95	0.20	4590	5.40	0.21	5500	6.00	0.21	6140		
C-2	1:2.04:5.39	0.47	6	3½	155.4	0.22	1.25	221	24x48				3370	5.20	0.19	3980	5.90	0.18	4650	5.80	0.16
	Wet-screened	0.47	3	3½	154.0	0.26	1.47	260	8x16	2810	4.65	0.18	3670	5.60	0.19	4250	5.70	0.18	5310	6.40	0.20
	Wet-screened	0.47	1½	3½	152.9	0.31	1.77	313	6x12	3660	5.20	0.21	5040	5.86	0.20	5500	5.90	0.21	6390	6.80	0.22
C-3	1:2.04:4.41	0.47	3	3¾	154.2	0.25	1.40	247	12x24				3860	5.50	0.18	4490	6.40	0.17	4930	6.40	0.16
	1:2.04:4.41	0.47	3	3¾	153.5	0.25	1.40	247	8x16	2920	5.00	0.17	3590	5.50	0.18	4380	5.70	0.17	5030	6.60	0.18
	Wet-screened	0.47	1½	3¾	152.3	0.30	1.70	300	6x12	3160	4.90	0.20	3920	5.70	0.22	4810	6.30	0.20	5770	6.80	0.23
C-4	1:2.04:3.62	0.47	1½	2¾	152.7	0.26	1.54	272	6x12	3060	5.10	0.22	3800	5.35	0.22	4760	6.00	0.20	5560	6.95	0.23
C-5	1:2.04:3.00	0.47	¾	3	150.3	0.29	1.66	293	6x12	3560	5.20	0.21	4500	5.50	0.22	5440	6.00	0.23	6260	6.70	0.26

TABLE 6—CONCRETE PROPERTIES AND TEST DATA FROM SERIES II

No.	Mix Data			Concrete Properties						Strength and Elastic Properties								
										7 Day Tests			28-Day Tests			90 Day Tests		
	Parts by Weight		Max. Size Agg.–In.	Slump–In.	U. Wt.– Lb. per cu. ft.	Abs. Vol. Paste Per Unit Vol. of Concrete	Cement Cont. Bbl. per cu. yd.	Water Cont.– Lb. per cu. yd.	Cylinder Size In.	U. Str.– Lb. p. sq. in.	Y. M. x 10^{-6}	P. R.	U. Str.– Lb. p. sq. in.	Y. M. x 10^{-6}	P. R.	U. Str.– Lb. p. sq. in.	Y. M. x 10^{-6}	P. R.
	C:S:G	W/C																
B-1	1:2.45:7.05	0.54	9	3	155.6	0.19	1.01	207	36x72				3090	5.10	0.18	3680	5.50	0.20
									24x48				2900	5.28	0.17	3340	5.60	0.20
									18x36				3030	5.50	0.18	3300	6.16	0.20
B-6	1:2.45:6.41	0.54	6	3	155.7	0.21	1.08	219	36x72				3180	5.77	0.20			
									24x48				3020	5.20	0.18	3540	5.60	0.18
									18x36				3140	5.80	0.17	3510	5.97	0.20
B-7	1:2.46:4.93	0.54	3	3¾	153.3	0.24	1.23	250	36x72				3310	4.97	0.17			
									24x48				3250	5.10	0.15	3910	5.22	0.17
									18x36				3300	5.27	0.18	3940	5.77	0.21
									12x24	2500	4.57	0.16	3190	5.37	0.21	3990	5.67	0.19
									8x16	2420	4.90	0.18	3470	5.40	0.19	4220	6.00	0.19
									6x12	2930	5.02	0.22	3610	5.32	0.22	3960	6.07	0.20
B-8	1:2.44:3.30	0.53	1½	5	150.5	0.28	1.49	297	36x72				3380	5.20	0.22			
									24x48				3400	4.72	0.20	4000	5.23	0.18
									18x36				3320			3840		
									12x24	2540			3590	4.15	0.17	4470	5.10	0.20
									8x16	2530			3730	4.83	0.18	4390	5.25	0.19
									6x12	2880	4.70	0.20	4050	5.40	0.21	4490	5.90	0.21
									3x6	2860	4.45	0.15	3650	5.25	0.18	4710	5.55	0.20
B-9	1:2.51:1.88	0.55	¾	8¼	146.9	0.35	1.78	368	24x48				3210	4.67	0.21	4170	4.37	0.16
									18x36				3360	4.95	0.19	4070	5.20	0.20
									12x24	2260			3380	4.68	0.20	4450	4.42	0.16
									8x16	2330	3.85	0.18	3470	4.36	0.16	4600	4.82	0.18
									6x12	2570	4.38	0.20	3750	4.67	0.22	4790	4.95	0.20
									3x6	2600	3.72	0.18	4020	4.40	0.18	5000	4.88	0.19
									2x4	2610			4060			4830		
B-10	1:2.52:0.75	0.54	⅜	9	143.2	0.41	2.14	435	18x36				3340	4.57	0.22	4010	4.70	0.22
									12x24	2320			3530	4.00	0.20	4370	4.27	0.17
									8x16	2210	3.12	0.17	3610	3.79	0.18	4590	4.20	0.19
									6x12	2400	3.33	0.21	3550	3.92	0.20	4740	4.63	0.21
									3x6	2600	3.37	0.18	3930	3.77	0.16	5000	4.42	0.19
									2x4	2640			4210			4910		


COMPRESSIVE STRENGTH-POUNDS PER SQ. IN.
4500
4000
3500
3000
2500
2000
1500
f'c = 0.47, Avg. = 3940 lbs/sq in, Mean Variation = 5.3%
f'c = 0.54, Avg. = 3420 lbs/sq in, Mean Variation = 3.9%
f'c = 0.66, Avg. = 2700 lbs/sq in, Mean Variation = 6.1%
Plotted values are from Table 5, all reduced to equivalent strength in 36 x 72-inch cylinders
Slump = 3 ins. ± for all mixes and sizes of aggregate
0 1 2 3 4 5 6 7 8 9
MAXIMUM SIZE OF AGGREGATE—INCHES

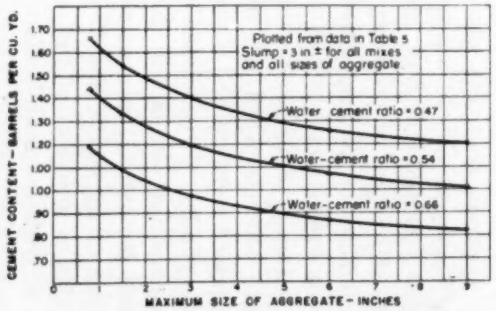
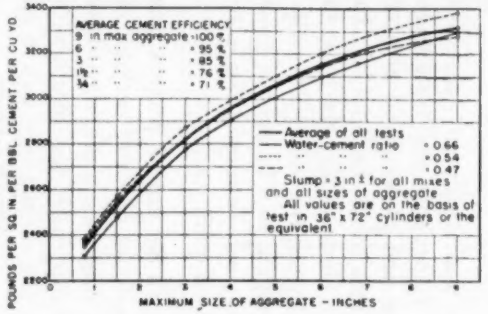


FIG. 6—EFFECT OF MAXIMUM SIZE OF AGGREGATE ON THE COMPRESSIVE STRENGTH OF CONCRETE AVERAGE OF 28 AND 90 DAY TESTS

FIG. 7—EFFECT OF MAXIMUM SIZE OF AGGREGATE ON THE CEMENT REQUIRED IN CONCRETE


COMPRESSIVE STRENGTH
POUNDS PER SQ. IN. PER BBL. CEMENT PER CU. YD.
AVERAGE CEMENT EFFICIENCY
9 in max. aggregate =100 %
6 " " " = 95 %
3 " " " = 85 %
1½ " " " = 76 %
¾ " " " = 71 %
Average of all tests
Water-cement ratio = 0.66
" " " = 0.54
" " " = 0.47
Slump = 3 in ± for all mixes
and all sizes of aggregate
All values are on the basis of test in 36" x 72" cylinders or the equivalent
0 1 2 3 4 5 6 7 8 9
MAXIMUM SIZE OF AGGREGATE - INCHES

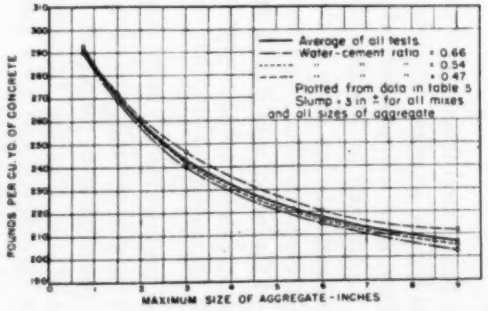

WATER CONTENT
POUNDS PER CU. YD. OF CONCRETE
Average of all tests
Water-cement ratio = 0.66
" " " = 0.54
" " " = 0.47
Plotted from data in table 5
Slump = 3 in ± for all mixes
and all sizes of aggregate
0 1 2 3 4 5 6 7 8 9
MAXIMUM SIZE OF AGGREGATE - INCHES

FIG. 8—EFFECT OF MAXIMUM SIZE OF AGGREGATE ON THE EFFICIENCY OF CEMENT IN CONCRETE AVERAGE OF 28 AND 90 DAY TESTS

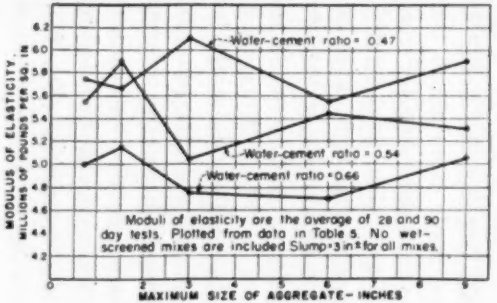
FIG. 9—EFFECT OF MAXIMUM SIZE OF AGGREGATE ON THE WATER REQUIRED IN CONCRETE

aggregate grading regardless of the richness of the mix. However, the efficiency is materially decreased as the maximum size of aggregate is decreased. The water required per unit of concrete decreases as the maximum size of aggregate increases. The water content curves are shown in Fig. 9.

It should not be inferred from Fig. 8 that 9-in. maximum aggregate is recommended for use in mass concrete. Experience and tests have demonstrated that more economical mixing and placing and greater uniformity can be effected by limiting the maximum size to 6 in. In some exceptional cases it may even be desirable to reduce the limit to 3 in. The more recent specifications issued by the Bureau of Reclamation have limited the maximum size of aggregate to 6 in. on the basis that the possible greater cement requirement will be more than compensated by the improvement in other desired qualities.

The facts that practically the same strength per unit of cement was obtained (Fig. 8) and essentially the same amount of water per unit of concrete was required (Fig. 9) for any given size of aggregate regardless of the richness of the mix are commensurate with the findings of Talbot, Richart, McMillan and Lyse since the consistency was in all cases maintained constant.[6,7,8]

The explanation of the effect of maximum size of aggregate on strength is not apparent from the available data. It seems logical that the size of aggregate should not greatly affect the compressive strength for the conditions of test used because the grading was changed only to the extent of dropping the successive sizes and the physical characteristics of the aggregates did not vary materially between the various sizes. Again referring to Fig. 6, it is seen that the same strength, within practical limits, was obtained for any maximum size of aggregate concrete when the consistency and water-cement ratio were maintained constant and if the mixes had all been tested in the same size of cylinder. It is pointed out that the conditions represented in Fig. 6 for any one of the three sets of mixes involve the following variable factors; (1) cement and water content, i. e. paste; (2) maximum size of aggregate, and to a certain extent, the grading and; (3) the ratio of diameter of test cylinder to maximum size of aggregate. It is entirely possible that the above factors are compensating and hence equal strengths are indicated. On the other hand, it is not clear in the case under discussion whether size of cylinder or the ratio of diameter to maximum size aggregate or both constitute variable factors. It is anticipated that additional data will soon be available from the Denver tests which will clarify the explanation.


Water-cement ratio = 0.47
Water-cement ratio = 0.54
Water-cement ratio = 0.66
Moduli of elasticity are the average of 28 and 90 day tests. Plotted from data in Table 5. No wet-screened mixes are included. Slump = 3 in. ± for all mixes.
MODULUS OF ELASTICITY, MILLIONS OF POUNDS PER SQ. IN.
MAXIMUM SIZE OF AGGREGATE - INCHES

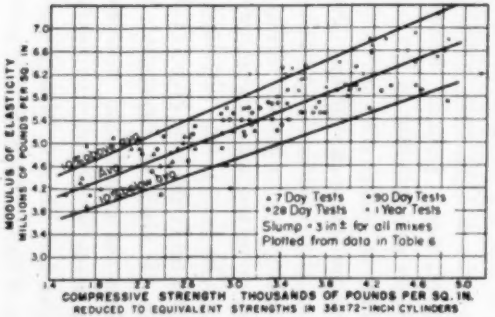
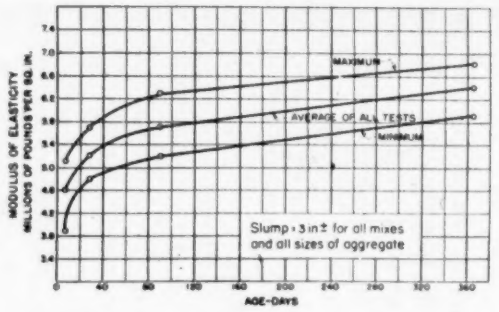

• 7 Day Tests • 90 Day Tests
• 28 Day Tests • 1 Year Tests
Slump = 3 in. ± for all mixes
Plotted from data in Table 6
MODULUS OF ELASTICITY, MILLIONS OF POUNDS PER SQ. IN.
COMPRESSIVE STRENGTH THOUSANDS OF POUNDS PER SQ. IN.
REDUCED TO EQUIVALENT STRENGTHS IN 36 x 72-INCH CYLINDERS

FIG. 10—EFFECT OF MAXIMUM SIZE AGGREGATE ON MODULUS OF ELASTICITY OF CONCRETE

FIG. 11—THE RELATION BETWEEN MODULUS OF ELASTICITY AND STRENGTH OF CONCRETE


MAXIMUM
AVERAGE OF ALL TESTS
MINIMUM
Slump = 3 in. ± for all mixes and all sizes of aggregate
MODULUS OF ELASTICITY, MILLIONS OF POUNDS PER SQ. IN.
AGE-DAYS

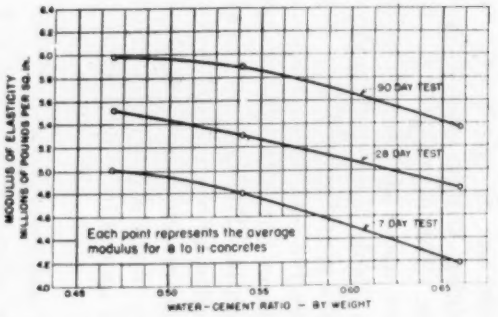

90 DAY TEST
28 DAY TEST
7 DAY TEST
Each point represents the average modulus for 8 to 11 concretes
MODULUS OF ELASTICITY, MILLIONS OF POUNDS PER SQ. IN.
WATER-CEMENT RATIO — BY WEIGHT

FIG. 12—THE RELATION OF MODULUS OF ELASTICITY TO AGE OF CONCRETE

FIG. 13—EFFECT OF WATER-CEMENT RATIO ON MODULUS OF ELASTICITY OF CONCRETE

Obviously the correct solution materially affects the apparent or real variation in compressive strength per unit of cement (cement efficiency) indicated in Fig. 8.

Fig. 10 shows the results obtained for Young's modulus plotted against maximum size of aggregate. Apparently the elasticity of concretes containing Arizona sand and gravel is not affected by the size of aggregate. This is probably due to the fact that the different gravel sizes have about the same elastic properties. The strength values listed in Table 5 were all reduced to equivalent values for 36 x 72-in. specimens and plotted against values for Young's modulus in Fig. 11. A trend toward higher modulus of elasticity with increasing strength is apparent but, as might be expected, the points are widely scattered and erratic. The results of tests tabulated in Table 5 indicate a consistent relation between modulus of elasticity and the age at test and also the water-cement ratio (Fig. 12 and 13). No effect on Poisson's ratio due to any of the above factors could be detected. The values obtained varied from 0.16 to 0.23 with an average value of 0.20 for all tests.

The effect of wet-screening—The tests reported in Table 6 also provide data for indicating the effect of wet-screening to successively smaller maximum sizes on various properties of mass concrete. It is again emphasized that the concretes used for fabricating the specimens included in this series of tests were not actually wet-screened from the 9-in. maximum full mix. Instead, tests were made to establish the average proportions and grading of the mixes that were actually wet-screened and these gradings and proportions were then used for the particular maximum size mix desired. Numerous tests were made to establish the fact that identical strengths were obtained whether the concretes were mixed with all sizes of aggregate and then screened to the required size or mixed with the larger sizes omitted. However, a marked difference in slump was obtained between the two methods. Wet-screening had no noticeable effect on the slump when the concrete was mixed with all sizes of aggregate included during the mixing process, but, when the equivalent wet-screened concretes were mixed with the larger sizes of aggregates omitted, a material increase in slump was obtained as the size of aggregate was decreased[10] (Fig. 15). The explanation of this phenomenon is not quite clear. A large number of experiments were conducted in investigating the matter which established the fact that the difference in slump was due to the action of the cobbles. Apparently the grinding action or pebble mill effect is not of material importance. The most satisfactory explanation seems

FIG. 14 — LONGITUDINAL COMPRESSOMETERS AND LATERAL EXTENSOMETERS MOUNTED ON A 36 BY 72-INCH CYLINDER

to be offered by the difference in the condition of packing or particle arrangement caused by the presence or absence of the cobbles during the mixing process. Fig. 16 shows the strength values from Table 6 plotted against maximum size of aggregate after wet-screening for two conditions of test, i. e., (1) with the size of test cylinder maintained constant at 36 x 72 in. for all sizes of aggregate and, (2) with the size of test cylinder varied such that the diameter is four times the maximum aggregate in all cases. It is apparent that the effect of wet-screening on the compressive strength of Boulder dam concrete is essentially the effect of cylinder size and that when wet-screened concrete is tested in the same size cylinder as the full mass mix, practically equal strengths are obtained. Furthermore, when all the values listed in Table 6 are reduced to equivalent strengths for 36 x 72-in. cylinders, the results are equal with a mean variation of only 3 per cent.[10]

Fig. 17 was prepared in accordance with the above deductions. It should be borne in mind that the values shown on this diagram represent average conditions. Individual values vary somewhat from the

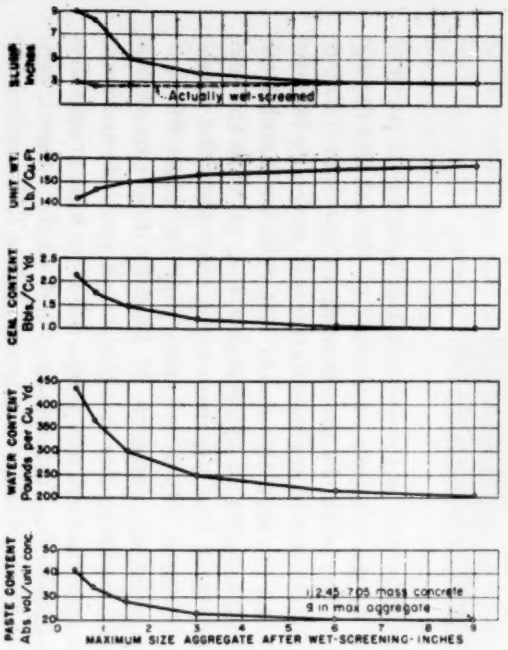


FIG. 15—EFFECT OF WET-SCREENING ON VARIOUS PROPERTIES OF CONCRETE

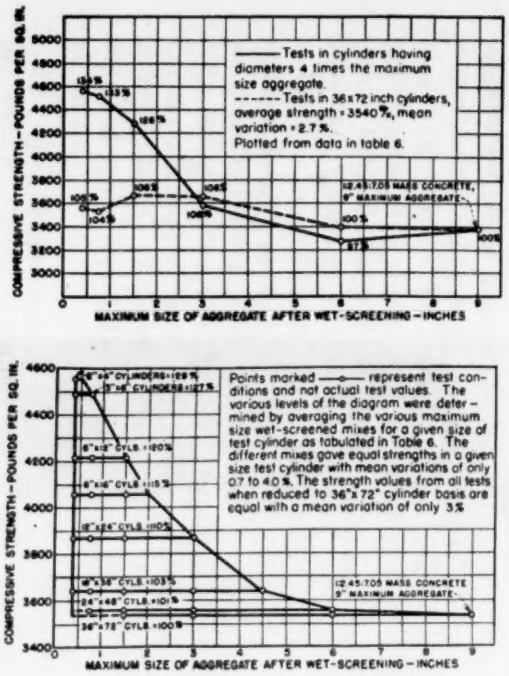

COMPRESSIVE STRENGTH – POUNDS PER SQ. IN.
——— Tests in cylinders having diameters 4 times the maximum size aggregate.
------ Tests in 36"x72" inch cylinders, average strength = 3540 #, mean variation = 2.7 %.
Plotted from data in table 6.
1:2.45:7.05 MASS CONCRETE, 9" MAXIMUM AGGREGATE
MAXIMUM SIZE OF AGGREGATE AFTER WET-SCREENING – INCHES
Points marked —o— represent test conditions and not actual test values. The various levels of the diagram were determined by averaging the various maximum size wet-screened mixes for a given size of test cylinder as tabulated in Table 6. The different mixes gave equal strengths in a given size test cylinder with mean variations of only 0.7 to 4.0 %. The strength values from all tests when reduced to 36"x 72" cylinder basis are equal with a mean variation of only 3%
1:2.45:7.05 MASS CONCRETE 9" MAXIMUM AGGREGATE

FIG. 16—EFFECT OF WET-SCREENING AND SIZE OF TEST CYLINDER ON THE COMPRESSIVE STRENGTH OF CONCRETE—AVERAGE OF 28 AND 90 DAY TESTS

FIG. 17—EFFECT OF WET-SCREENING ON THE COMPRESSIVE STRENGTH OF CONCRETE —AVERAGE OF 28 AND 90 DAY TESTS

indicated percentages, however, the mean variations are very small as noted on the diagram.

The wet-screened data obtained in connection with the series of tests represented in Table 5 check the factors indicated in Fig. 16 within reasonable limits. The average ratio from Table 5 between 1½-in. wet-screened concrete tested in 6 x 12-in. cylinders and 9-in. mass concrete tested in 36 x 72-in. cylinders for all ages and mixes is 129 per cent as compared with 126 per cent from Fig. 16. The factor of 120 shown in Fig. 17 is somewhat lower; however, it has been found that the latter value applied for a large group of tests not reported in this paper.

The data presented herein indicates that 9-in. Boulder dam mass concrete when tested in 36 x 72-in. cylinders will yield 77 per cent of the strength indicated by the field control test, i. e., 1½-in. wet-screened concrete in 6 x 12-in. cylinders.

Values for Young's modulus from Table 6 are plotted against maximum size of aggregate after wet-screening in Fig. 18. No explanation is offered for the results indicated in this diagram as compared with those shown in Fig. 10.

Effect of curing condition—A large number of tests have been made in connection with cement studies for Boulder dam which show the comparative strength of concrete cured in standard fog rooms maintained at 70°F and under conditions simulating those obtaining in the interior of massive structures (mass curing). Some of the average data are given in Tables 7 and 8 and are shown graphically in Fig. 19 and 20.

Standard curing relates to specimens stored unsealed in fog rooms maintained at a temperature of 70°F. Mass curing refers to specimens sealed in thin metal containers and stored in specially designed adiabatic calorimeter rooms on a rising temperature cycle from various starting temperatures for a period of 28 days and in rooms maintained at 70°F thereafter. The temperature cycle of the calorimeter rooms for each test run was automatically controlled by the temperature rise of a 17 x 17-in. cylinder of mass concrete containing one barrel of cement per cu. yd. and Boulder dam aggregates.[9]

Fig. 19 shows the comparative strengths for standard fog and mass curing from three starting or placing temperatures for a limited number of three different types of portland cements. These diagrams are self explanatory and require no particular comment except to call attention to the apparent permanent loss in strength due to mass curing from a high starting temperature with standard cements. The low heat cements represented in this group are not typical of those actually

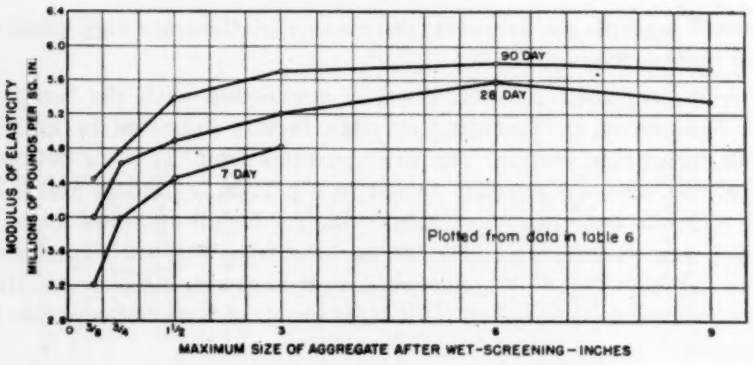

MODULUS OF ELASTICITY
MILLIONS OF POUNDS PER SQ. IN.
90 DAY
28 DAY
7 DAY
Plotted from data in table 6
MAXIMUM SIZE OF AGGREGATE AFTER WET-SCREENING - INCHES

FIG. 18—EFFECT OF WET-SCREENING ON THE MODULUS OF ELASTICITY OF CONCRETE

used in Boulder dam as may be seen by comparison with Fig. 20. The superior strength development of the laboratory group of cements which include both low-heat and high-heat compositions is also of interest.

Fig. 20 illustrates the strength development for two types of curing for a large number of representative cements for each of three types. The acceleration of early strength development is typical for all types of cements, but in all cases the later strengths of specimens mass cured are less than the strengths of specimens standard cured.

Fig. 21 shows the effect of placing temperature on the consistency of concretes and mortars as measured by the slump test when the water-

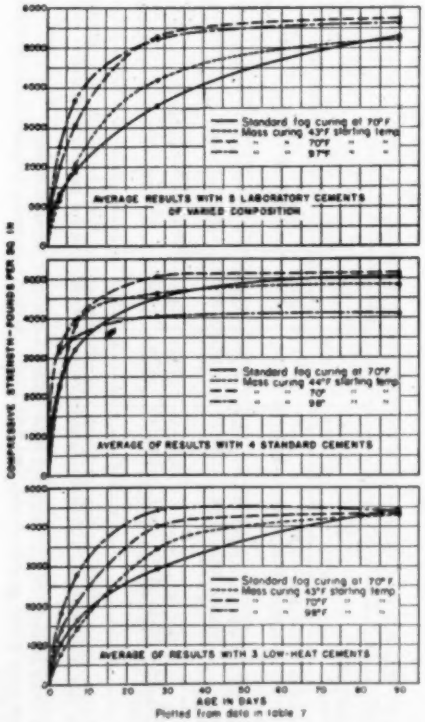
TABLE 7—EFFECT OF PLACING TEMPERATURE ON THE COMPRESSIVE STRENGTH OF MASS CURED CONCRETE

Type of Cement	Curing	3 Days	7 Days	28 Days		90 Days	Notes
Laboratory cements of varied composition	Std. Fog—70°F.	1280**	1900	3640	3180*	5310	Average results with five cements
	Mass—43°F. start	1140**	2040	4200	3450*	5230	
	Mass—70°F. start	1960**	3030	5290	3880*	5750	
	Mass—97°F. start	2360**	3640	5320	4070*	5440	
Standard portland cements	Std. Fog—70°F.	2270**	3250	4530	3790*	5160	Average results with four cements
	Mass—44°F. start	2200**	3810	4620	3840*	4830	
	Mass—70°F. start	3280**	3980	4790	3610*	5140	
	Mass—98°F. start	3220**	3500	4040	3380*	4140	
Low-heat portland cements	Std. Fog—70°F.	1000**	1570	2970	2500*	4480	Average results with three cements
	Mass—43°F. start	450**	1350	3470	3240*	4340	
	Mass—70°F. start	1340**	2060	4010	3380*	4450	
	Mass—98°F. start	1640**	2750	4400	3440*	4170	

Each value represents the average of 3 each 2 by 4-inch plastic mortar, 3 by 6-inch, ¾-inch maximum concrete and 6 by 12-inch, 1½-inch maximum concrete specimens for each cement unless otherwise noted.

*One 18 by 36-inch, 6-inch maximum mass concrete specimen for each cement.

**Only 2 by 4-inch plastic mortar and 3 by 6-inch, ¾-inch maximum concrete specimens included in 3 day tests.


COMPRESSIVE STRENGTH—POUNDS PER SQ IN
Standard fog curing at 70°F
Mass curing 43°F starting temp
" " " 70°F " "
" " " 97°F " "
AVERAGE RESULTS WITH 5 LABORATORY CEMENTS OF VARIED COMPOSITION
Standard fog curing at 70°F
Mass curing 44°F starting temp
" " " 70° " "
" " " 98° " "
AVERAGE OF RESULTS WITH 4 STANDARD CEMENTS
Standard fog curing at 70°F
Mass curing 43°F starting temp
" " " 70°F " "
" " " 98°F " "
AVERAGE OF RESULTS WITH 3 LOW-HEAT CEMENTS
AGE IN DAYS
Plotted from data in Table 7

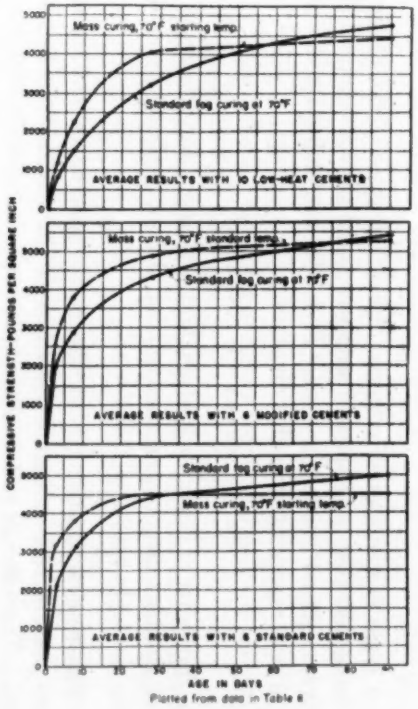

COMPRESSIVE STRENGTH—POUNDS PER SQUARE INCH
Mass curing, 70°F starting temp
Standard fog curing at 70°F
AVERAGE RESULTS WITH 10 LOW-HEAT CEMENTS
Mass curing, 70°F starting temp
Standard fog curing at 70°F
AVERAGE RESULTS WITH 6 MODIFIED CEMENTS
Standard fog curing at 70°F
Mass curing, 70°F starting temp
AVERAGE RESULTS WITH 6 STANDARD CEMENTS
AGE IN DAYS
Plotted from data in Table 6

FIG. 19—EFFECT OF STARTING TEMPERATURE ON THE COMPRESSIVE STRENGTH OF MASS CURED CONCRETE

FIG. 20—EFFECT OF CURING CONDITION ON THE COMPRESSIVE STRENGTH OF CONCRETE

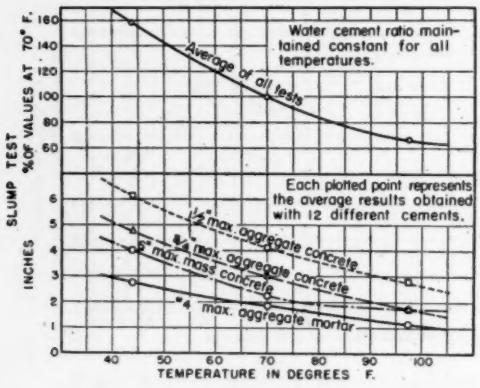

SLUMP TEST
% OF VALUES AT 70° F.
INCHES
Water cement ratio maintained constant for all temperatures.
Average of all tests
Each plotted point represents the average results obtained with 12 different cements.
1½" max. aggregate concrete
¾" max. aggregate concrete
6" max. mass concrete
¼" max. aggregate mortar
TEMPERATURE IN DEGREES F.

FIG. 21—EFFECT OF PLACING TEMPERATURE ON THE CONSISTENCY OF CONCRETE AT THE MIXER

cement ratio is maintained constant. For purposes of conversion, it may be stated that a change in water-cement ratio of about 0.03 by weight is required to effect a change of one inch in slump for the conditions of test represented in this diagram.

SUMMARY OF FINDINGS

The following statements and evaluations apply for concretes containing Boulder dam aggregates and for the methods and conditions used in the tests previously discussed.

1. The indicated strength of the same concrete when tested in various sizes of test cylinders decreases as the size of specimen is increased. The average relative strengths were found to be 100 per cent for 6 x 12-in., 108 per cent for 2 x 4 in., and 84 per cent for 36 x 72-in. cylinders.

TABLE 8—EEFFCT OF CURING CONDITION ON THE COMPRESSIVE STRENGTH OF CONCRETE

Type of Cement	Curing	3 Days	7 Days	28 Days		90 Days	Notes
Boulder Dam Low-Heat Portland	Std. Fog—70°F. Mass—70°F. start	840** 1320**	1530 2270	3250 4050	2740* 3650*	4780 4460	Average results with 10 cements
Modified Normal Portland	Std. Fog—70°F. Mass—70°F. start	1980** 2530**	2890 3810	4310 4880	....	5420 5290	Average results with 6 cements
Standard (High-heat) Portland	Std. Fog—70°F. Mass—70°F. start	2120** 3170**	3160 3830	4410 4510	3550* 3550*	4990 4510	Average results with 6 cements

Each value represents the average of 3 each 2 by 4-inch plastic mortar, 3 by 6-inch, ¾-inch maximum concrete and 6 by 12-inch, 1½-inch maximum concrete specimens for each cement unless otherwise noted.

*One 18 by 36-inch, 6-inch maximum mass concrete specimen for each cement.

**Only 2 by 4-inch plastic mortar and 3 by 6-inch, ¾-inch maximum concrete specimens included in 3 day tests.

2. The elastic properties of concretes are not affected by the size of test cylinder.

3. For consistent results, the diameter of the test cylinder should be equal to or greater than four times the maximum size of aggregate included in the concrete to be tested.

4. Maximum size of aggregate has no appreciable effect on the compressive strength of concretes when the proportions of coarse aggregate are varied to maintain the water-cement ratio and slump constant and when the concretes are all tested in the same size of test cylinder.

5. The cement required for concretes of equal slump and water-cement ratio increases as the maximum size of aggregate is decreased.

6. About the same cement efficiency or unit strength per unit of cement content is obtained from concretes of equal consistency and a given aggregate grading regardless of the richness of the mix.

7. The cement efficiency decreases as the size of aggregate is decreased when the consistency and size of test cylinder are maintained constant.

8. The maximum size of aggregate recommended for use in mass concrete is limited to 6 in.

9. About the same total amount of water is required in concrete for a given slump and aggregate grading regardless of the richness of the mix.

10. The relationship between strength and cement-water ratio by by weight is essentially a straight line variation.

11. The tests have not completely established the effect of maximum size of aggregate on the elastic properties of concretes.

12. In general, the modulus of elasticity of concretes increases as the strength is increased.

13. The variations for Young's modulus with varying water-cement ratio or age are similar to the variations for compressive strength.

14. Poisson's ratio is not noticeably affected by any of the variable factors included in the investigations under consideration.

15. Wet-screening mass concrete to successively smaller aggregate sizes does not materially affect the compressive strength when the tests are all made in the same size of test cylinder.

16. Wet-screening does not affect the slump of concrete as compared with the consistency of the original full mix.

17. The slump of concrete reproportioned equivalent to wet-screened concrete and mixed with the larger sizes of aggregate omitted during the mixing process increases materially as the size of aggregate

is decreased. This effect is due to the action of the cobbles in the mixer. The strength of concrete is the same for both types of mixing.

18. Mass concrete containing 9-in. maximum size aggregate and tested in 36 x 72-in. cylinders will yield 67 to 85 per cent (average 77 per cent) of the corresponding values obtained with 1½-in. maximum wet-screened concrete tested in 6 x 12-in. cylinders.

19. Mass curing as previously defined and as compared with standard fog curing increases strength of concrete at the early ages and decreases the strength at the later ages.

20. The effect of curing condition on the compressive strength of concrete depends somewhat upon the type of cement.

Other information and data of value not discussed in this paper have been obtained in connection with the Boulder dam investigations. The complete data will be presented and discussed in detail in a series of Government bulletins which will be published during the current year. In conclusion, it is desired to repeat, "——that much needed information pertaining to the characteristics of mass concrete has been obtained, making possible a more scientific and efficient selection, combination and manipulation of materials for massive construction work."[3]

ACKNOWLEDGMENTS

All activities of the Bureau of Reclamation are supervised by Dr. Elwood Mead, Commissioner of Reclamation, Washington, D. C. All engineering and construction is under the general direction of R. F. Walter, Chief Engineer, with headquarters in Denver. All research and design work is under the direction of J. L. Savage, Chief Designing Engineer.

The investigations described herein were conducted under the general supervision of B. W. Steele, Designing Engineer on Dams.

Special acknowledgments are due to W. H. Price and B. E. Bussell, Assistant Engineers for assistance in assembling the data.

REFERENCES

1. Steele, B. W., "Mass Concrete Research for Hoover Dam," JOURNAL AMER. CONCRETE INST., March-April, 1933, *Proceedings*, Vol. 29, p. 305.
2. Ruettgers, Arthur, "Mass Concrete as Affected by Size of Aggregate and Related Factors," JOURNAL AMER. CONCRETE INST., Sept.-Oct., 1933, *Proceedings*, Vol. 30, p. 27.
3. Blanks, R. F., "Boulder Dam Cement and Concrete Studies," *Engineering News-Record*, Vol. 113, No. 21, p. 648 (Nov. 22, 1934).
4. Blanks and Vidal, "Apparatus for Fabricating and Testing Mass-Concrete Cylinders," *Proceedings, A. S. T. M.*, Vol. 33, Pt. II, p. 538 (1933).

5. Vidal, E. N., "Development of Apparatus for Measuring Elasticity of Mass Concrete," JOURNAL AMER. CONCRETE INST., Sept.-Oct., 1933, *Proceedings*, Vol. 30, p. 41.
6. Talbot and Richart, "The Strength of Concrete—Its Relation to the Cement, Aggregates and Water," *University of Illinois Bulletin No. 137*, Oct., 1923.
7. McMillan, F. R., "Basic Principles of Concrete-Making," *Engineering News-Record*, Apr. 11, 1929.
8. Lyse, Inge, "Cement-Water Ratio by Weight Proposed for Designing Concrete Mixes," *Engineering News-Record*, Nov. 5, 1931; "Simplifying Design and Control of Concrete Mixes," *Engineering News-Record*, Feb. 18, 1932.
9. Meissner, H. S., "Development of Large Calorimeter Rooms and Automatic Temperature Controls for Adiabatic Curing of Mass Concrete," JOURNAL AMER. CONCRETE INST., Sept.-Oct., 1933, *Proceedings*, Vol. 30, p. 21.
10. Powers, T. C., "Concrete Studies at the Bull Run Dam," AMERI. CONCRETE INST., *Proceedings*, Vol. 25, 1929, p. 388.
11. Powers, T. C., "Concrete Studies at the Bull Run Dam," *Proceedings*, AMERI. CONCRETE INST., Vol. 25, p. 388, 1929.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by July 1, 1935.*

Tests of Mesnager Hinges*

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AND A. H. STANG‡

I. INTRODUCTION

Seven reinforced concrete specimens with hinges (articulations) of the Mesnager type[1] were fabricated and tested at the National Bureau of Standards. The specimens represented portions of reinforced concrete arches of the barrel type. Determinations of strength as well as measurements of the stiffness of the hinges when subjected to compressive, shearing and rotational deformations, were included in the tests.

II. ACKNOWLEDGMENTS

The specimens were designed by Lieut. Com. Ben Moreell, Bureau of Yards and Docks, Navy Department, Washington, D. C. They were fabricated and tested at the National Bureau of Standards. The cement for the specimens was donated by the International Cement Corporation; the reinforcement by the Truscon Steel Co., and the Hudson Supply & Equipment Co.; the latter firm also lent an electric hammer for use in vibrating the concrete.

III. THE SPECIMENS WITH HINGES

1. *Types*—The details of the specimens are shown in Fig. 1, and views of the specimens after test are shown in Fig. 2 and 3. One of the end blocks of each specimen may be considered to represent a short length of the barrel or rib of a reinforced concrete arch. The other would then represent either a portion of an abutment, or, for a crown hinge, another short length of the arch barrel or rib. In each specimen the distance between the end blocks was 4 in.

The end blocks of all specimens contained both longitudinal and web reinforcement, as shown in Fig. 1. In the central portion of the specimens the intradosal and extradosal longitudinal bars crossed the mid-plane of the arch, at an angle of 30 deg., the hinge being formed

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[1]The functions of hinges (flexible joints), methods of use and results of previous tests of Mesnager hinges are to be described by Lieut. Com. Ben Moreell in a paper to be presented at the 31st Annual Convention, New York, February 19-21, 1935, and later published in the Journal of the American Concrete Institute.

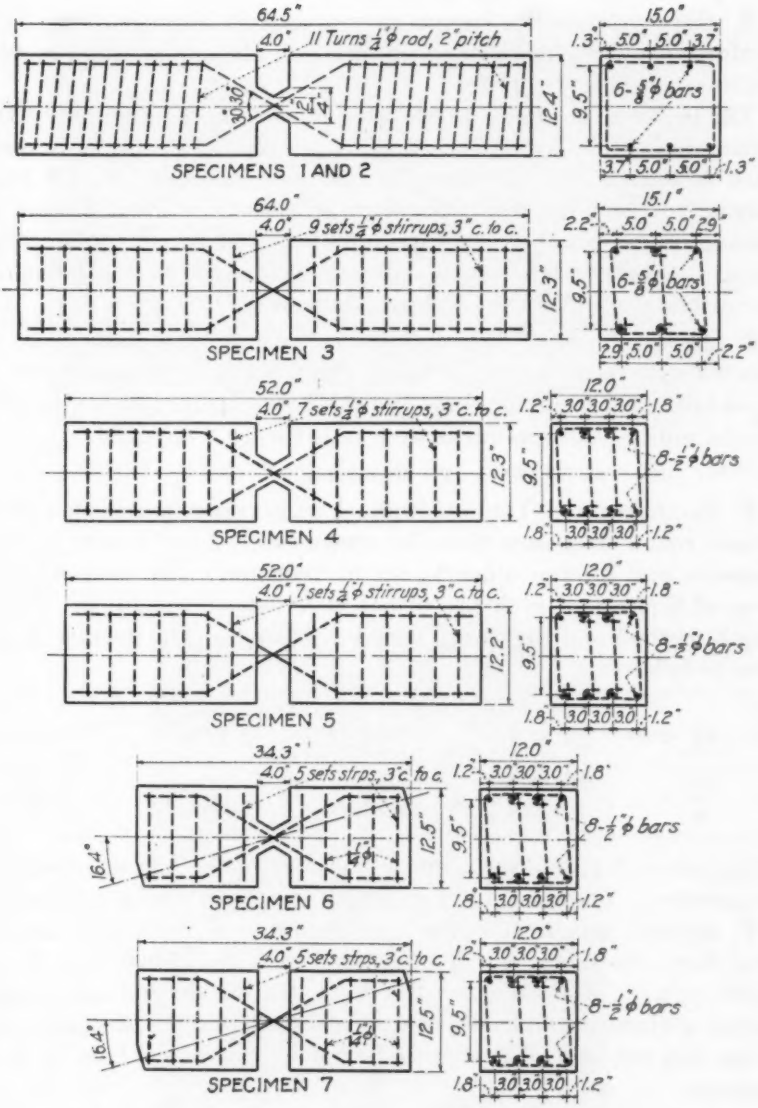


FIG. 1—DETAILS OF THE SPECIMENS WITH HINGES

by the crossing of these bars. The bars were bare between the end blocks in specimens 3, 5 and 7; in the others they were embedded in mortar, the least covering being ½ in. thick.

2. *Construction*—The specimens were cast in a wooden form. The reinforcement for a specimen was bent, assembled, and wired in place before concreting was begun.

The materials for the concrete were proportioned by weight. After mixing in a drum mixer, the entire batch was dumped into a metal-lined box from which it was shoveled into wheelbarrows for transporting to the form, and then shoveled into the form. During the placing of the concrete the form was vibrated by means of a light electric hammer. The specimens were fabricated in the laboratory during the months of October and November.

3. *Aging*—Within 2 hours after the concrete was placed, it was covered with damp burlap. When 1 day old the specimens were removed from the forms and kept in the laboratory at room temperature and humidity. All specimens were tested when 3 days old.

IV. MATERIALS

1. *Reinforcement*—The longitudinal reinforcement consisted of deformed round steel bars; those for specimens 1, 2 and 3 were ⅝ in. in diameter and for the other ½ in. in diameter. The cross-sectional areas of 5 pieces of each size of bar were determined by the weight-length method and they were tested in tension. The average results were as follows:

Nominal Diameter of Bar	Cross Sectional Area	Modulus of Elasticity	Yield Strength From Drop of Beam	Tensile Strength
Inch	Inch²	Lb/in.²	Lb/in.²	Lb/in.²
⅝	0.296	29,600,000	43,600	73,500
½	.193	29,500,000	46,700	72,200

One piece, 3.3 in. long, from a ⅝-in. diameter bar was tested in compression. The modulus of elasticity was 29,800,000 p. s. i. and the yield strength was 40,100 p. s. i. in compression; in tension another piece from the same bar gave, respectively, 29,700,000 p. s. i. and 40,900 p. s. i. The differences between the tensile and compressive moduli of elasticity and yield stress are too small to be of significance.

The web reinforcement consisted of plain round steel bars ¼ in. in diameter.

2. *Concrete*—A high early strength cement and Potomac river sand and gravel were used in the concrete. The maximum size of the gravel was 1 in. Three 6 x 12-in. cylinders were made with concrete from each specimen, the concrete being consolidated by rodding and by

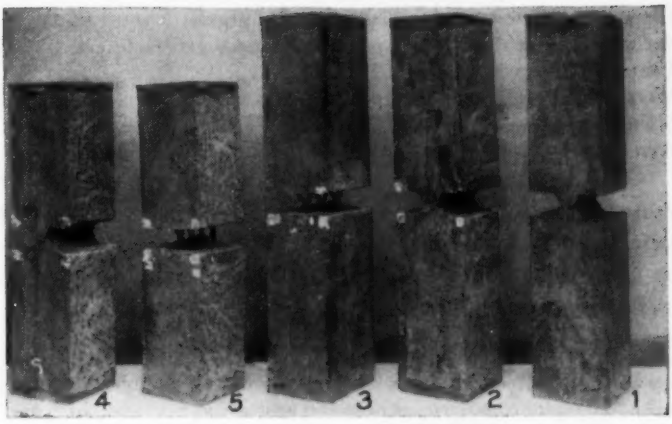


FIG. 2—VIEW OF SPECIMENS 1 TO 5 AFTER TESTING

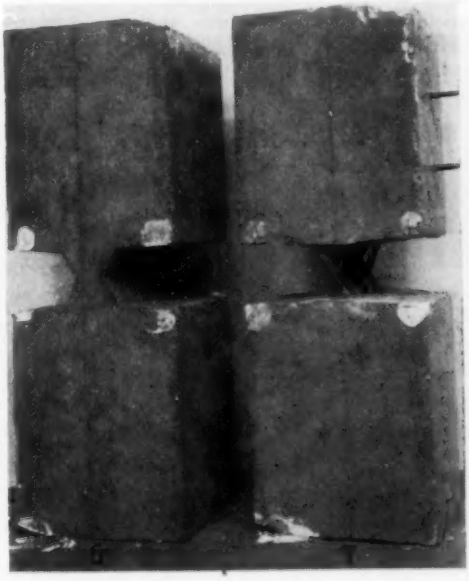


FIG. 3—VIEW OF SPECIMENS 6 AND 7 AFTER TESTING

vibrating the molds with a light electric hammer. When 1 day old the cylinders were placed in damp storage. The data on the concrete are as follows:

Proportions by weight of cement, sand and gravel	1:2.17:3.84
Sacks (94 lb. each) of cement per cu. yd. of concrete	5.85
Cement-water ratio by weight	2.31
Water-cement ratio, gal. per sack	4.89
Slump, inch	0.4
Average compressive strength of 6 x 12-in. cylinders at 3 days, p. s. i.	3630

The mortar used in the central 4 in. length of specimens 1, 2, 4 and 6 was made by mixing cement, sand, and water in the same proportions as were used in the concrete. The average compressive strength of five 6 x 12-in. mortar cylinders at 3 days was 3250 p. s. i.

V. METHODS OF TESTING

The specimens were tested in compression: Specimen No. 1 under central loading; No. 2, 3, 4 and 5 under combined bending and central loading; and No. 6 and 7 under combined shearing and central loading. The first type of loading was approximately equivalent to tangential thrust at a hinge of an arch, the second to a forced rotation at the hinge combined with a tangential thrust, and the third, to a thrust with its line of action making an angle of 16.4 deg. with the arch axis, without rotation. For the first two types the thrust was equal to the load, and there was no shearing force; for the third, the thrust was 0.959 of the load, and the shear was 0.282 of the load. The methods of applying the testing forces are illustrated in Fig. 4.

To prepare a specimen for testing, it was first placed centrally in the testing machine. A plastic mortar of quick-hardening cement and sand was then applied to the bearing faces of the end blocks; after which most of the mortar was squeezed out with as little pressure as possible by closing the gap between the end blocks and the platens of the testing machine. The mortar was allowed to age for 1 day before a test was made.

Specimen No. 1 was tested in a vertical testing machine; the others were tested in a horizontal machine. Those tested in a horizontal position were supported on carriages below, as illustrated in Fig. 5, except that before a test steel balls between steel plates were interposed between the carriage and specimen in order to minimize frictional resistance to movements of the specimen in a horizontal direction.

Rotations and compressive deformations of the hinges were measured by means of 4 dial gauges supported on steel yokes clamped to the concrete blocks, as illustrated in Fig. 5; the relative rotation of the end blocks was taken as the rotation of the hinge and the shortening of the gap as the compressive deformation of the hinge. A fifth dial gauge (not shown) was used to measure the shearing deformations of hinges 6 and 7. The readings of this gauge, after being corrected for relative rotation between the end blocks, indicated the relative displacement at the center of the hinge of an extension of the mid-plane of one block to that of the other block. These corrected readings are hereinafter called shearing deflections.

The loads were applied in increments small enough to provide data for a determination of relations between loads and deformations. The time from the start of the loading until failure ranged from 30 to 50 minutes.

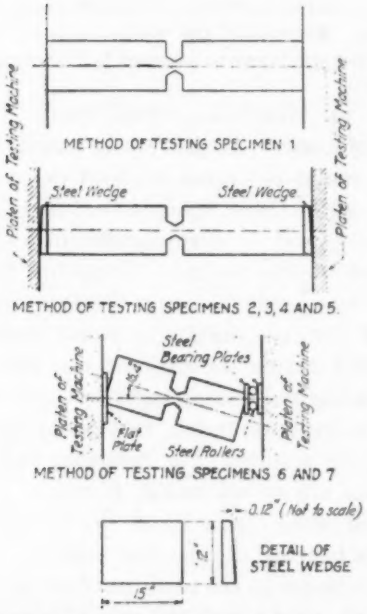


FIG. 4—METHODS OF LOADING SPECIMENS. PLAN VIEWS OF SPECIMENS AND DETAILS OF WEDGES

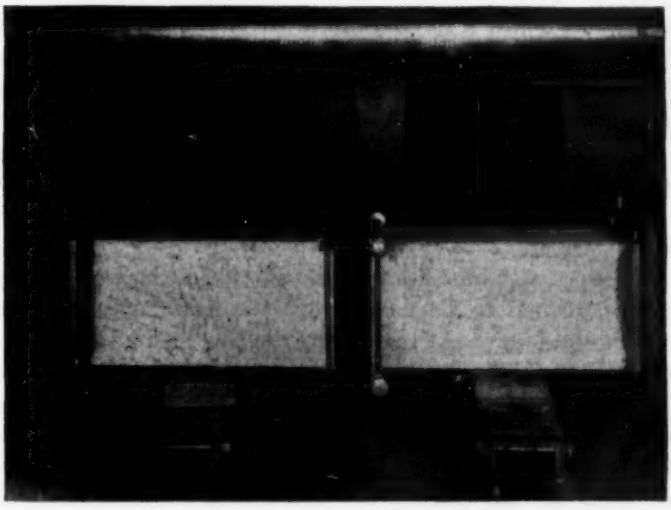


FIG. 5—SPECIMEN IN TESTING MACHINE

During a test the specimens were supported on 1-in. diameter steel balls between steel plates to minimize frictional resistance to motion.

Specimen No. 3 was loaded to 25,000 lb., the load released, the wedges reversed and load increased to failure. Reversal of the wedges reversed the direction of bending of the hinge. All the other specimens were tested by increasing the loads until failure occurred.

VI. RESULTS OF THE TESTS

1. *Eccentricities of Loads*—As previously noted, it was intended that specimens 6 and 7 would be tested without bending. Some bending, however, occurred with these specimens. Similarly the maximum rotations of hinges 3 and 5 were significantly different from that caused by the taper of the wedges. Probably these hinges rotated slightly as pressure was applied during the capping, which resulted in the capped bearing surfaces making a small angle with the platens when the load applied during the capping was released. However, as the rotations were measured during each test and as none was greatly different from the amounts intended, these deviations do not seriously impair the value of the results. The rotations and shearing and compressive deformations are shown in Fig. 6 and 7.

2. *Behavior of Specimens Under Load*—Specimens 2, 3, 4 and 5 were tested with the ends bearing against the wedges. The rotations were approximately proportional to the loads until about ¾ of the maximum rotation had occurred; after which the rate of increase of rotation with load decreased, until the wedges appeared to be fully in contact with the platens. After full contact, the angle at the hinge remained nearly constant.

In specimens 2 and 4, having mortar over the hinge bars, the initial rotations took place about an axis a little less than 1 in. from the initial axis of the hinge, causing an increase in the average distance between the faces of the concrete blocks. On the side of the hinge in which tensile stresses were caused by the rotations, tensile cracks appeared in the mortar covering. The cracks began to close when the rotations ceased and appeared to be fully closed before the maximum loads were reached.

For the hinges with bare bars, the shortening of the gaps was approximately proportional to the loads until the latter exceeded one-third of the maximum loads. At high loads the shortening increased a little more rapidly than the loads, the shortening increasing rapidly just before failing. Even at loads less than one-third the maximum, the shortening of the gaps was much greater than could be attributed to the elastic deformations in the bars and in the concrete within the gage lengths. Undoubtedly part of the shortening was caused by slipping of the bars in the concrete near the faces of the blocks. The first sign of failure of the specimens with bare bars was an increase in the rate of compression and a scaling of the bars. The loads continued to increase,

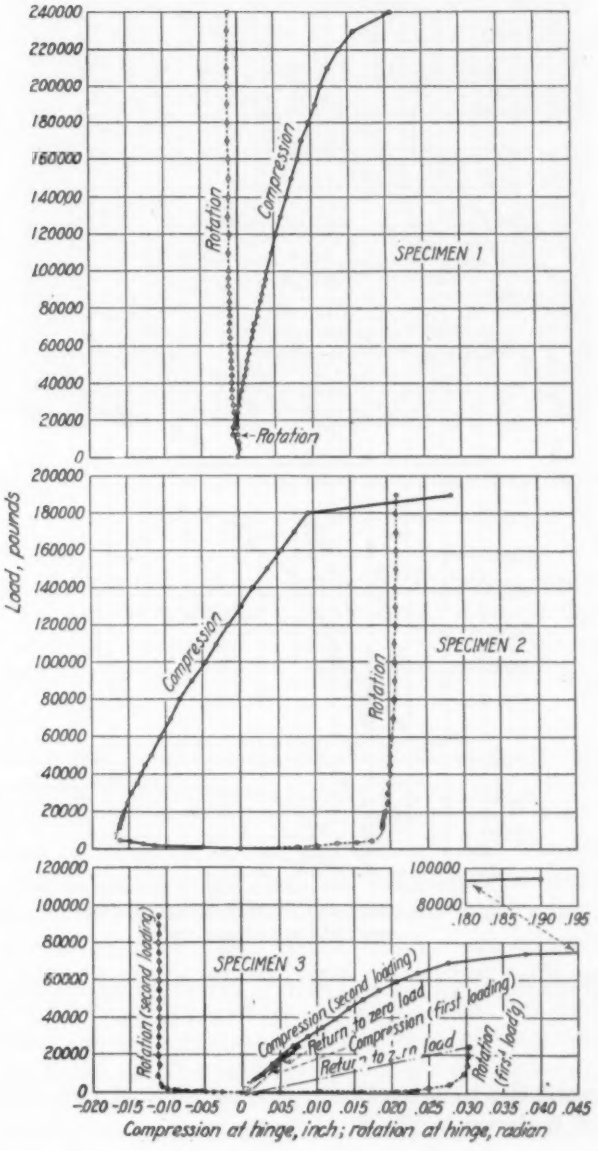


FIG. 6—ROTATIONS AND COMPRESSIVE DEFORMATIONS OF SPECIMENS 1, 2 AND 3

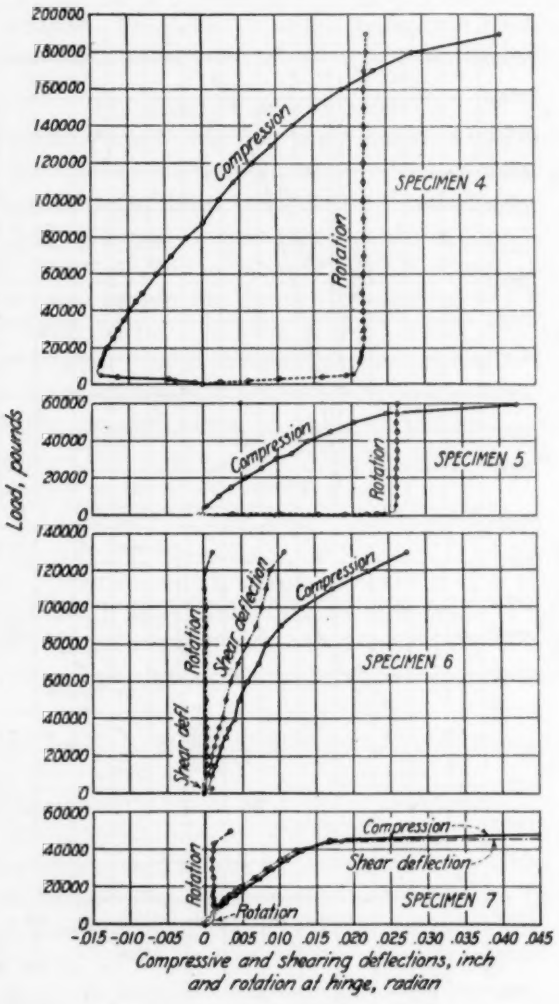


FIG. 7—ROTATIONS AND COMPRESSIVE AND SHEARING DEFORMATIONS OF SPECIMENS 4 TO 7 INCLUSIVE

however, until the bars buckled. A close view of two of the hinges after failure is shown in Fig. 8.

Cracks appeared in the concrete blocks of all specimens except 5 and 7. In specimens 1, 2 and 4 the blocks began to split before the maximum load was reached, the cracks starting at the hinge end of the block and extending toward the other ends. The cracks in specimen

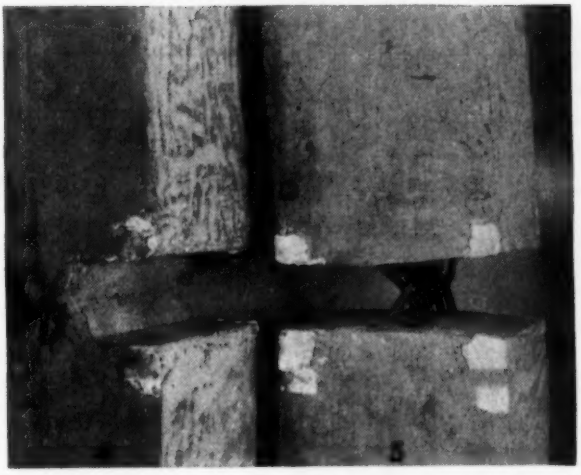
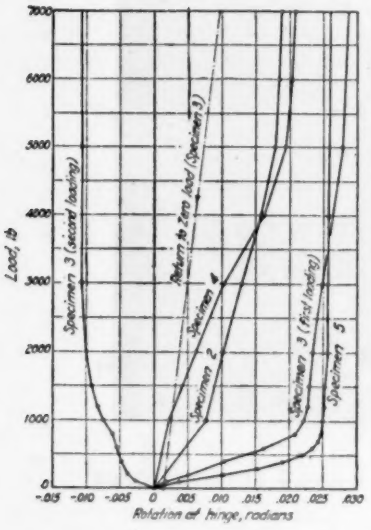


FIG. 8—VIEW OF SPECIMENS 4 AND 5 AFTER FAILURE

No. 3 were similar but did not appear until the maximum load was reached. The cracks in specimen No. 6 appeared before the maximum load was applied and tended to follow the plane of the more highly stressed set of hinge bars. The locations of some of the cracks may be seen in Fig. 2 and 3.

3. *Resistance to Rotation*—During the first part of the loading with wedges between the bearing faces of a specimen and the platen of the testing machine, only one edge of a wedge made contact with a platen. The bending moment applied to the specimen was therefore approximately 6 lb. in. per pound of load. Although the moment arm of the load could not be determined with precision and the loads were in some cases too small to be measured accurately, a rough measure of the resistance of the hinges to rotation or flexure at low loads was obtained from the rotations. The rotations of the hinges at low loads as shown in Fig. 9 and the data relating to the stiffness of the hinges are summarized in Table 1. The table gives also for the two specimens with bare bars, calculated values of moments causing a rotation of 0.01 radian. The moments were computed by means of the commonly used equation for the flexure of straight homogeneous beams of uniform cross section, considering that all bars were bent to circular arcs of the same curvature, and that the effective length of a bar was the total length of the portion not completely embedded. That is, effective



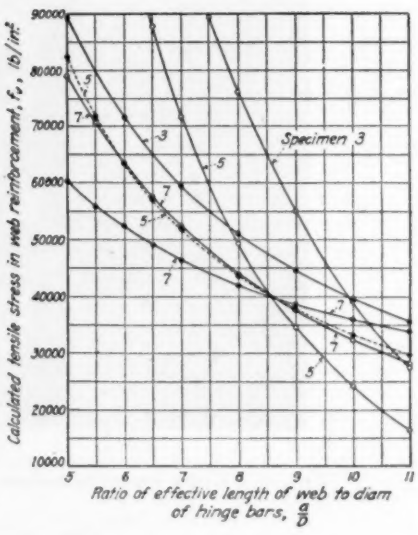

Calculated tensile stress in web reinforcement, f_w, lb/in.²
Specimen 3
Ratio of effective length of web to diam of hinge bars, $\frac{a}{D}$

FIG. 9 (Left)—ROTATIONS OF THE HINGES
FIG. 10—CALCULATED STRESSES IN WEB REINFORCEMENT.
a = effective length of web and D = diameter of hinge bars

length $= l = \frac{4}{\cos 30°} + D \tan 30°$, inches in which D = diameter of bar, inch.

TABLE 1—ROTATIONAL STIFFNESS OF THE HINGES

Specimen No.	Mortar Over Bars	Width of Specimen	Hinge Bars				Bending Moment Per Pair of Bars Lb. In. Per 0.01 Radian	
			Number of Bars	Diameter D	Length[1] l	Slenderness Ratio l/r	Actual	Computed
		Inches		Inch	Inches			
2	Yes	15.0	6	0.614	—	—	5,700	—
3	No	15.1	6	.614	4.97	32.4	770	830
4	Yes	12.0	8	.496	—	—	3,800	—
5	No	12.0	8	.496	4.91	39.6	320	355

[1] $l = \frac{4}{\cos 30°} + D \tan 30°$ = length of bar not completely embedded in concrete.

As shown by the data of Table 1 the resistance of the hinges to bending was increased by the mortar covering, the stiffness of those with mortar being 7.5 to 11.5 times as great as for those with bare bars

TABLE 2—STRENGTHS OF THE SPECIMENS

Specimen No.	Mortar Over Bars	End Blocks		Longitudinal Reinforcement				Rotation at Yield Load[1] ϕ	Yield Load[1]		Load at First Rapid Yielding		Maximum Load	
		Width	Thickness	Number of Bars	Nominal Diameter	Total Area	Ratio, Area of Steel to Area of Concrete		Total	Per Pair of Bars	Total	Per Pair of Bars	Total	Per Pair of Bars
		Inches	Inches		Inch	In.^{2}		Radian	Lb.	Lb.	Lb.	Lb.	Lb.	Lb.
1	Yes	15.0	12.4	6	5/8	1.78	0.0095	0.001	213,000	71,000	230,000	77,000	246,000	82,000
2	Yes	15.0	12.4	6	5/8	1.78	.0095	.021	93,000	31,000	180,000	60,000	193,000	64,400
3	No	15.1	12.3	6	5/8	1.78	.0096	.011	60,000	20,000	70,000	23,300	95,000	31,600
4	Yes	12.0	12.3	8	1/2	1.54	.0105	.022	64,000	16,000	180,000	45,000	191,000	47,800
5	No	12.0	12.2	8	1/2	1.54	.0105	.026	35,000	8,800	55,000	13,700	60,000	15,000
6	Yes	12.0	12.5	8	1/2	1.54	.0103	.000	92,000	23,000	100,000	25,000	138,000	34,500
7	No	12.0	12.5	8	1/2	1.54	.0103	.001	40,000	10,000	40,000	10,000	52,000	13,000

[1]Yield strength (set in compression = 0.003 inch) by the "set method" of E8-33, 1933, Book of A. S. T. M. Standards.

but otherwise similar. The stiffness of both types of hinges, however, was small compared with that of the end blocks.

The calculated resistance to bending of the hinges with bare bars was about 10 per cent greater than that estimated from loads and measured movements. For these calculations the moments of inertia of the deformed bars were taken as equal to those of cylindrical bars of the same weight per unit length. Inasmuch as the actual moments of inertia were less than for cylindrical bars of equal average cross-sectional area, this accounts in part for the discrepancy between the actual and the calculated resistance to bending. Another possible cause of the discrepancy is that the moment arm of the loads may have been less than 6 in., as assumed.

4. *Resistance to Thrust and Shear*—The data on the strength of the specimens are summarized in Table 2. The effect of rotation on the resistance to thrust of hinges with mortar covered bars is indicated by the results for No. 1 and 2. Specimen 2 was rotated 0.02 radian more than No. 1 and the strength was about 20 per cent less than that of No. 1. Mortar covering over the bars increased the thrust resistance from 2 to 3.2 times. This is seen by comparing the maximum load for No. 2 with that for No. 3, No. 4 with No. 5, and No. 6 with No. 7. The data for No. 4 to 7 show that the maximum loads were less when the angle of thrust was 16.4° without rotation than with a rotation of 0.022 to 0.026 radian without shear.

Roughly approximate values of stresses in the bare bars of some hinges may be calculated by means of simple formulas which neglect the effects of deformations in the concrete blocks and the effects of lateral deflections of the bars on the moments produced by the thrusts. These formulas are discussed in the Appendix. The maximum stress in the bare bars of specimens 3 and 5 is given by equation (12) of the Appendix which may be written (in which T and ϕ are positive and $V = O$):

$$\sigma_m = \frac{T}{2A}\left[\frac{1}{c\left(1 + 12t^2\left(\frac{r}{l}\right)^2\right)} + \frac{l}{sr}\left(\frac{1}{1 + \frac{1}{12t^2}\left(\frac{l}{r}\right)^2}\right)\right]$$

$$+ \frac{V}{2A}\left[\frac{1}{S\left(1 + \frac{12}{t^2}\left(\frac{r}{l}\right)^2\right.} - \frac{l}{cr}\left(\frac{1}{1 + \frac{t^2}{12}\left(\frac{l}{r}\right)^2}\right)\right] + \frac{2Er\phi}{l} \quad \ldots\ldots (1)$$

in which T = thrust load per pair of bars,
V = shearing load per pair of bars,
ϕ = rotation at hinge, radians (considered as positive here),
$l = 4.61 + 0.577D$ = effective length of bar,
r = radius of gyration of bar,
A = cross sectional area of one bar,
E = modulus of elasticity of steel bars,
θ = angle between hinge bars at gap and mid-plane of specimen,
$c = \cos \theta$,
$s = \sin \theta$,
$t = \tan \theta$,

For specimen 7, equation (13) applies, which is the same as the foregoing except for a change in the signs of the second terms within the brackets.

Maximum stresses at yield load calculated by means of equation (1) are given in Table 3. For comparison there are given also values calculated by means of the following equation:

$$\bar{s} = \frac{1}{2A}\left(\frac{T}{C} + \frac{V}{S}\right) + \frac{2Er\phi}{l} \quad \ldots\ldots (2)$$

in which the stresses caused by T and V are calculated as for a pin connected truss (see equation (14) of Appendix). The calculated maximum stress at the yield load for specimen No. 7, which was rotated by 0.001 radian, is not greatly different from the yield strength of the bars; for specimens 3 and 5, the calculated stresses are greater, being the larger the calculated bending stresses. This is consistent with theory and the results of tests on struts subjected to combined bending and thrust.

TABLE 3—CALCULATED MAXIMUM STRESSES AT YIELD LOADS IN HINGES WITH BARE BARS

Specimen No.	Area of One Bar A	Slenderness Ratio l/r	Rotation at Hinge ϕ	Forces Per Pair of Bars at Center of Hinge for Yield Load		Calculated Maximum Stresses in Bars at Yield Load	
				Thrust T	Shear V	Equations (1)	Equation (2)
	In.²		Radian	Lb.	Lb.	Lb/in.²	Lb/in.²
3	0.296	32.4	0.011	20,000	0	68,000	59,000
5	.193	39.6	.026	8,800	0	70,000	65,000
7	.193	39.6	.001	9,600	2,800	47,000	45,000

5. *Resistance of Web of Reinforced Concrete End Blocks*—As previously noted, cracks appeared in the concrete blocks of specimens 1, 2, 4 and 6 before the maximum loads had been applied. A crack appeared in one of the blocks of No. 3 as the load reached the maximum; none

was seen in No. 5 and 7. Probably the maximum loads on No. 1, 2 and 4 would have been greater if these had contained more effective web reinforcement. It was evident that the web reinforcement in No. 5 and 7 was enough to prevent failure in the web prior to failure of the hinge bars.

A method for calculating the actual stresses in the web reinforcement is not available, but some useful information may be obtained by comparing the test results with values calculated on an empirical basis. Tensile forces in the ribs, perpendicular to the arch axis, are caused by thrust loads on a hinge. Some of the thrust load of each bar of a hinge is transferred to the concrete through bond, causing compressive stresses in the web parallel with the arch axis and tensile stresses perpendicular thereto. The latter are resisted partly by the concrete and partly by the web reinforcement. Obviously the web reinforcement beyond a certain distance from the hinge will not be effective in resisting these tensile forces. The test results afford data for obtaining a rough estimate of how much of the web reinforcement might be considered as effective in resisting the tensile forces in the web.

It is shown in the Appendix that the reactions from a pair of bars in a hinge with bare bars caused a tensile force in the end blocks which was approximately

$$F_1 = \frac{T}{2} \tan 30°$$

This force was augmented in specimens 6 and 7 by the effect of the shearing force, V, on the hinge. It is commonly assumed[2] that the tensile force in the web due to the shearing force is given roughly by:

$$F_2 = \frac{Va}{jd}$$

in which a = length of web along axis of end blocks considered effective in resisting tensile force in web, inches,

j = a factor, here taken as 0.9,

d = distance from face of concrete to mid-plane of longitudinal bars near opposite face, inches.

Stresses in the web reinforcement of specimens 3, 5 and 7 as calculated by means of formulas of two different types are shown in Fig. 10. The values represented by the open circles were calculated by means of the formula:

$$f_v = [F_1 + F_2 - 200ab] \frac{1}{A_v}$$

[2] Technologic Paper BS 20, 390 (1926); T314. Bulletin University Illinois 166 (1927).

in which f_v = tensile stress in web reinforcement, lb/in.^{2} and A_v = cross-sectional area of web reinforcement in the length a and width b, sq. in. This equation is based upon the assumption that the concrete resists a tensile stress of 200 lb/in.^{2} or a force of 200 ab lb.

The values represented by the filled circles were computed by means of the formula:

$$f_v = \frac{F_1 + F_2}{0.005ab + A_v}$$

which implies that the resistance of the concrete is the same as an area of reinforcement equal to $0.005ab$.

The width of the crack in the web of Specimen 3 indicated that the web reinforcement had been stressed nearly to or slightly beyond the yield stress, whereas the stresses in specimens 5 and 7 were not great enough to show cracks in the concrete. For $a/D = 8$ or 9 (D being the diameter of the hinge bars), the stresses as calculated by the formulas were about equal to or somewhat greater than the yield strength for specimen 3 and were less than the yield strength for specimens 5 and 7. Although the data are too meager to form a satisfactory basis for design, they indicate that the web reinforcement was adequate when, in the following equation, f_v was less than the yield strength of the steel.

$$f_v = \frac{\frac{T}{2} \tan \theta + \frac{Va}{jd}}{0.005ab + A_v},$$

a being taken as $8D$.

VII. SUMMARY OF RESULTS

The chief object of the tests was to determine the strengths and stiffness of the particular hinges tested. It was not possible in this work systematically to change one condition at a time as would be desirable in a general study of the structural behavior of Mesnager hinges. Nevertheless, the results summarized here are believed to provide information which will be of assistance in estimating their structural value.

1. The rotations at the hinges were approximately proportional to the applied bending moments. For the hinges with bare bars, the resistance to rotation was about the same as for straight bars having a length equal to that of the exposed hinge bars. The rotational stiffness of the hinges with mortar covering over the bars was 7.5 to 11.5 times as great as that of hinges with bare bars.

2. For the hinges which were subjected to rotations, the shortening of the hinge gaps at loads exceeding one-third of the maximum was not proportional to the loads but increased more rapidly than the loads. Both the observations and the calculations indicated that some portions of the bars of each hinge yielded at a load considerably less than the maximum and that there was some slipping of the bars in the concrete.

3. At least one of the concrete end blocks was cracked in all but two of the specimens, in some cases before the maximum load had been applied. Stresses in the web reinforcement, calculated by means of an empirical equation, gave a rough indication of the tendency of the concrete blocks to split. The observations indicated that web reinforcement was most effective when some of the bars were near the hinge gap.

4. For the hinges with bare bars, the calculated stresses in the bars at the maximum loads considerably exceeded the yield strength, as is usual for short struts subjected to combined axial and transverse loads. Hinges with mortar covering over the bars withstood loads 2 to 3.2 times as great as those with bare bars.

APPENDIX

Calculation of Stresses in Bare Bars of Hinges

Although an accurate general analysis of the stresses in the bars of a Mesnager-type hinge is hardly possible, the calculation of stresses in a structure which resembles the simplest form of Mesnager hinge may be useful. The simplest type of hinge contains bars in two planes only. If the deformation in the concrete is neglected, a pair of contiguous bars may be considered as the diagonals of a truss with the ends of each bar joined to the ends of the others by means of a rigid member (concrete). This is represented by Fig. 11. The length of a bar is taken as $l = \frac{q}{\cos\theta} + D \tan\theta$, D being the diameter of the bar. M, T and V are, respectively, the moment thrust and shear at the center of the hinge. The moment at the center of the hinge is equal to the moment at the face of the concrete to the right (Fig. 11) plus the product of the shear and one-half the width of the hinge gap.

Presumably the action of the bars in the assumed structure would represent quite well that in the bare bars of the hinges for stresses within the elastic range. A slight slipping of the bars in the concrete, if of equal magnitude for both bars, would cause a greater shortening of the distance q under axial thrust than would be estimated but would not cause a serious error in calculations for the stresses in slender bars. Unequal slipping of the bars would cause the proportion of load carried by the two bars to be different from that assumed. Errors from this cause would tend to become greater, the greater the average stress in the bars and would be larger for stiff than for slender bars. As the shearing force V causes stresses of opposite sign in the two bars, this force would always tend to cause slipping in opposite directions. It appears, therefore, that the foregoing assumption may be applied to calculate approximate stresses, within the proportional limits, in hinges with bare bars, provided the bars

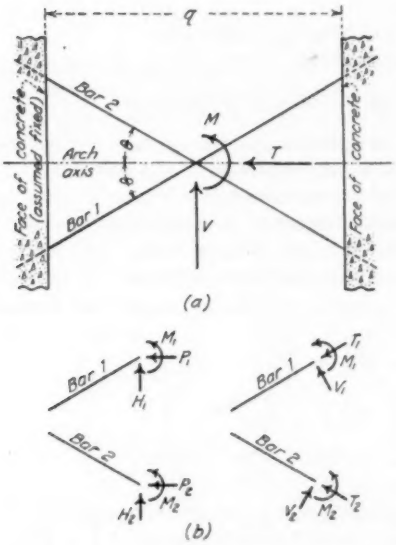


FIG. 11—EQUIVALENT SYSTEMS OF FORCES. (A) FORCES ACTING AT CENTER OF HINGE, (B) FORCES AT RIGHT ENDS OF BARS

are not too stiff and provided the shearing force is not too large in comparison with the thrust and moment.

The stresses in the bars of the assumed structure (Fig. 11), if expressed in terms of the forces acting at the ends of the individual bars, can be calculated by means of well known methods.[3] But an evaluation of the stresses in terms of the forces acting on the hinge results in equations which are too cumbersome for practical use. The equations are much simplified if the principle of superposition is assumed to be applicable. Although the resulting equations are fair approximations for only a limited range of slenderness ratios, the useful range may be increased somewhat by the use of approximate corrections intended to take account of the effect of buckling action on increasing the moments and stresses.[4]

If the deformations of the bars are assumed to be proportional to the forces acting on the hinge, the distribution of the forces to the bars and hence the moments and stresses may be calculated readily by means of Castigliano's method. The following notation will be used.

M, T, V = moment, thrust, and shear per pair of bars, respectively, acting at center of hinge (Fig. 11).
M_1 and M_2 = moments at right ends of bars 1 and 2, respectively.
P_1 and P_2 = forces parallel to T at right ends of bars 1 and 2, respectively.
H_1 and H_2 = forces parallel to V at right ends of bars 1 and 2, respectively.
$s = \sin \theta$
$c = \cos \theta$

[3]See, for example, Strength of Materials, by S. Timoshenko, D. VanNostrand Co., Inc., and Theory of Structures, by A. Morely, Longmans, Green & Co.
[4]See footnote [3].

t = tan θ
q = length of hinge gap along axis
D = diameter of bar

$l = \frac{q}{c} + Dt$ = effective length of bar
$r = D/4$ = radius of gyration of bar
A = cross-sectional area of bar = $4\pi r^2$
I = moment of inertia of bar = $4\pi r^4$
E = modulus of elasticity of material of bar
ϕ = rotation at hinge, radians
x = distance along bar from right end.

With the foregoing notation the thrusts, shears, and moments on the bars are:

$$\left.\begin{aligned} \text{Thrust bar 1, } T_1 &= P_1c - H_1s \\ \text{bar 2, } T_2 &= P_2c + H_2s \end{aligned}\right\} \quad (1)$$

$$\left.\begin{aligned} \text{Shear bar 1, } V_1 &= P_1s + H_1c \\ \text{bar 2, } V_2 &= -P_2s + H_2c \end{aligned}\right\} \quad (2)$$

$$\left.\begin{aligned} \text{Moment bar 1, } M_{1x} &= M_1 + P_1sx + H_1cx \\ \text{bar 2, } M_{2x} &= M_2 - P_2sx + H_2cx \end{aligned}\right\} \quad (3)$$

The strain energy in bars 1 and 2, respectively, neglecting the small amount due to shearing deformations is given by the following:

$$\left.\begin{aligned} W_1 &= \frac{T_1^2 l}{2AE} + \frac{1}{2EI}\int_o^l M_{1x}^2\, dx \\ W_2 &= \frac{T_2^2 l}{2AE} + \frac{1}{2EI}\int_o^l M_{2x}^2\, dx \end{aligned}\right\} \quad (4)$$

If the concrete is assumed to be rigid, the rotation, ϕ, of the right ends of both bars with reference to the left ends, assumed to be fixed, will be equal; hence

$$\left.\begin{aligned} &\frac{\partial W_1}{\partial M_1} = \frac{\partial W_2}{\partial M_2} = \phi \\ &\text{The difference between the displacements of the ends of bars 1 and 2 along } P_1 \text{ and } P_2\text{, respectively, is given by} \\ &\frac{\partial W_1}{\partial P_1} - \frac{\partial W_2}{\partial P_2} = ls\ sin\ \phi \\ &\text{and the difference between their displacements along } H_1 \text{ and } H_2\text{, respectively, is} \\ &\frac{\partial W_1}{\partial H_1} - \frac{\partial W_2}{\partial H_2} = -\ ls\ vers\ \phi \end{aligned}\right\} \quad (5)$$

Substituting W_1 and W_2 from equations (4) in the first of equations (5) and using the obvious equalities

$$\left.\begin{aligned} T &= P_1 + P_2 \\ V &= H_1 + H_2 \\ M &= M_1 + M_2 + \frac{V\,l\,c}{2} + (2P_1 - T)\frac{ls}{2} \end{aligned}\right\} \quad (6)$$

the following equation is found for the relation between the moment and the rotation of the hinge:[5]

[5]Harris Epstein, designing engineer, Bureau of Yards and Docks, derived equations (7), (8), and (9) independently. Instead of using the conditions expressed by equations (5), he expressed the strain energy in terms of the forces at the fixed ends of the bars and, using the method of least work, found equations (7), (8), and (9) directly.

$$\phi = \frac{Ml}{2EI} \Bigg\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The equations which express the relations between the forces (T and V) at the center of the hinge and the thrusts (T_1 and T_2) and the shears (V_1 and V_2) on the bars may be simplified somewhat by neglecting the effects of the rotations of the hinges. For most practical cases the thrusts (T_1 and T_2) and the shears (V_1 and V_2) on the bars which are caused by small rotations of the hinge will be negligible in comparison to those caused by the forces T and V. Neglecting the effects of rotation on the thrusts T_1 and T_2 and the shears V_1 and V_2 is equivalent to setting $\sin \phi = \phi$ and $\text{vers } \phi = 0$ in equations (5). By substituting W_1 and W_2 from equations (4) in the last two of equations (5) with the foregoing approximations and using equations (6), the following equations are found:[6]

for the axial thrusts on bars 1 and 2:

$$\left.\begin{aligned} T_1 &= \frac{T}{2c}K - \frac{V}{2s}L \\ T_2 &= \frac{T}{2c}K + \frac{V}{2s}L \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which $K = \dfrac{1}{1 + 12t^2\left(\dfrac{r}{l}\right)^2}$ and $L = \dfrac{1}{1 + \dfrac{12}{t^2}\left(\dfrac{r}{l}\right)^2}$

and for the shears on bars 1 and 2

$$\left.\begin{aligned} V_1 &= \frac{V}{2c}R + \frac{T}{2s}Q \\ V_2 &= \frac{V}{2c}R - \frac{T}{2s}Q \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which $R = \dfrac{1}{1 + \dfrac{t^2}{12}\left(\dfrac{l}{r}\right)^2}$ and $Q = \dfrac{1}{1 + \dfrac{1}{12t^2}\left(\dfrac{l}{r}\right)^2}$

[6] If the sine and versine terms of equations (5) are retained, the following equations are found in lieu of (8) and (9):

$$T_1 = \frac{T}{2c}K - \frac{V}{2s}L + \frac{AE}{2}\left[Q\,vers\,\phi - tR\,(\phi - sin\,\phi)\right]$$

$$T_2 = \frac{T}{2c}K + \frac{V}{2s}L + \frac{AE}{2}\left[Q\,vers\,\phi + tR\,(\phi - sin\,\phi)\right]$$

$$V_1 = \frac{V}{2c}R + \frac{T}{2s}Q - \frac{AE}{2}\left[\frac{Q}{t}vers\,\phi + tR\,(\phi - sin\,\phi)\right]$$

$$V_2 = \frac{V}{2c}R - \frac{T}{2s}Q + \frac{AE}{2}\left[\frac{Q}{t}vers\,\phi - tR\,(\phi - sin\,\phi)\right]$$

Now R and Q are always less than 1. Assuming $\phi < 0.05$ radian, $\frac{l}{\gamma} > 20$ and $15° < \theta < 45°$ and using the approximations $\sin \phi = \phi - \frac{\phi^3}{6}$ and $\text{vers } \phi = \frac{\phi^2}{2}$ the last terms in the foregoing equations reduce to one of the following:

$$\frac{AE\phi^2}{4}\left[Q \pm \frac{tR\phi}{3}\right] \text{ and}$$

$$\mp \frac{AE\phi^2}{4}\left[\frac{Q}{t} \pm \frac{tR\phi}{3}\right]$$

It is readily seen that these terms are small in comparison to values of T_1, T_2, V_1, and V_2 which may be considered critical in design, but they may under certain conditions constitute a large proportion of thrust or shear on a bar. Errors caused by the use of the approximations included in equations (8) and (9) are always small in calculations of stresses, as the equation for stresses contains a term with the first power of ϕ.

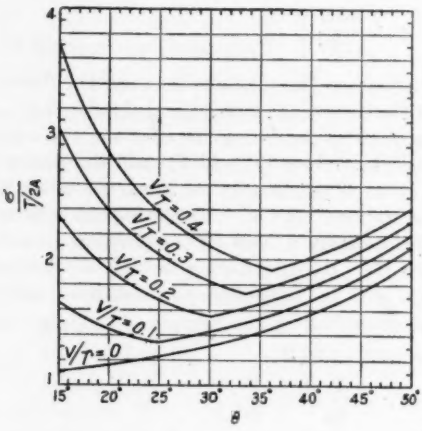


FIG. 12—RELATIONS BETWEEN θ AND THE MAXIMUM STRESS WITH $l/r = 50$ AND $\phi = 0$

σ = maximum stress calculated by means of equation (12) or (13)
T = thrust per pair of hinge bars
V = shear per pair of hinge bars
A = cross sectional area of one bar
θ = one-half the angle between the two hinge bars

The bending moments in the bars are:

$$\left.\begin{aligned} M_{1x} &= \frac{EI\,\phi}{l} + V_1\,(x - l/2) \\ M_{2x} &= \frac{EI\,\phi}{l} + V_2\,(x - l/2) \end{aligned}\right\} \qquad (10)$$

and the maximum stresses at any transverse section are:

$$\frac{T_1}{A} + \frac{M_{1x}}{2\pi r^3} \text{ and } \frac{T_2}{A} + \frac{M_{2x}}{2\pi r^3}$$

The maximum stress in a bar will be at $x = 0$ or $x = l$. It is apparent also, because of symmetry, that if the forces M, T, and V are constant as to magnitude but variable as to sign, the maximum stress at one end of a bar will be equal to the maximum stress possible in either bar. Selecting bar 2 at $x = l$, we find for the maximum stress:

$$\sigma_m = \frac{T_2}{A} \pm \left[2E\,\phi \left(\frac{r}{l}\right) + \frac{V_2 l}{4\pi r^3} \right] \qquad (11)$$

Substituting from (8) and (9) in (11), we have

$$\sigma_m = \frac{T}{2A}\left[\frac{K}{c} + \frac{Q}{s}\frac{l}{r}\right] + \frac{V}{2A}\left[\frac{L}{s} - \frac{R}{c}\frac{l}{r}\right] - \frac{2Er\,\phi}{l} \qquad (12)$$

in which T and V are taken as positive and ϕ as negative and which applies when $\frac{V}{T} < \frac{Q}{Rt}$. Also

$$\sigma_m = \frac{T}{2A}\left[\frac{K}{c} - \frac{Q}{s}\frac{l}{r}\right] + \frac{V}{2A}\left[\frac{L}{s} + \frac{R}{c}\frac{l}{r}\right] + \frac{2Er\,\phi}{l} \qquad (13)$$

in which T, V, and ϕ are taken as positive and which applies when $\frac{V}{T} > \frac{Q}{Rt}$.

The relations between θ and the maximum stress in a bar for various values of the ratio, $\frac{V}{T}$, all for $l/r = 50$, and $\phi = 0$ are indicated in Fig. 12.

It will be noted that as l/r becomes large, K and L approach unity and R and Q approach zero; then V_1 and V_2 approach zero and the maximum stress caused by the forces T and V approaches $\frac{1}{2A}\left(\frac{T}{c} + \frac{V}{s}\right)$, ...(14)

the same as for a pin connected truss, that due to bending remaining $\frac{2Er\phi}{l}$.

Inasmuch as the effect of the deflections of the bars on the stresses produced by axial forces has not been included in equations (12) and (13), the actual stresses would tend to be greater than calculated values. The difference would tend to become large as the thrust load on a bar approached the same order of magnitude as the buckling load. For a hinge subjected only to a thrust along the axis of the arch, the distributions of the forces between the bars would not be affected by the lateral deflections of the bars produced by the axial loads on them. The deflection of a bar would be symmetrical about the point of inflection at its mid-point. Moreover, the buckling loads for the bars would be relatively high and the deformations of each bar would be similar to that of a strut with ends restrained against rotation but forced to undergo a relative lateral displacement. For hinges subjected to a shearing force, buckling deformations in the bars would cause the distribution of the forces between the bars to be different from that assumed in the preceding sections, the effect being to decrease the loads on the more highly stressed set of bars. The lowest possible buckling load would be obtained with a combination of thrust, shear, and moment on the hinge, which caused the curvature of the more highly stressed bar to be of constant sign. For any particular loading the effects of buckling action on the stresses may be estimated[7] and it can be shown that the calculated stresses are less for values of l/r between 50 and 100 than for l/r less than 50.

In calculating the tensile stresses in the web reinforcement, use was made of two different forms of empirical equations which have been derived to express the relation between shearing force on the web of a reinforced-concrete beam and the stresses in the web reinforcement. The forces H_1 and H_2 (Fig. 11) cause a tensile force in the reinforced-concrete blocks of the hinge. Using the approximations included in equations (8) and (9), the magnitude of this force is given by

$$F_1 = \tfrac{1}{2}(H_2 - H_1) = \frac{Tl}{2}\left[\frac{1 - 12\left(\frac{r}{l}\right)^2}{1 + 12l^2\left(\frac{r}{l}\right)^2}\right] \quad \text{.....(15)}$$

or for $l/r > 20$ and $15° < \theta < 45°$

$$F_1 = \frac{Tl}{2} \text{ (approx.)}.$$

[7]See footnote [2]; also Trans. Am. Soc. Civil Engr., Vol. 95 (1931), p. 1221 et seq.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by July 1, 1935.*

Discussion of a Paper by Raymond E. and Harmer E. Davis:

Bonding of New Concrete to Old at Horizontal Construction Joints*

CONVENTION DISCUSSION

W. G. Chace (Consulting Engineer, Toronto): Almost all Professor Davis' conclusions seem to coincide with what one might arrive at if he sat down after a little experience in concrete making, analyzed these conditions immediately and wrote down the answers. I have had experience in placing some hundreds of thousands of yards of concrete in 30 years or more and through all the different "styles." In 1902, at Niagara Falls, the concrete was dry—the practice was general in that regard for some few years thereafter. Then mechanical means of placing concrete were introduced requiring a wetter mix, and that became the style. Then Duff Abrams showed us that we were losing a lot of money and we went back to the drier mix. Now Professor Davis advocates a half-inch slump, which means that we must spend a lot of money to place it. In 1909-10 I built a dam across the Winnipeg River—dry concrete dam, in sections 45 ft. long; these might vary from 6 to 13 ft. thick, the height about 15 or 16 ft. Overflow sections experienced later on an ice thrust which ruptured the structures at one of the horizontal construction joints and for a length of about 500 ft., including 10 or more of these 45 ft. sections. Our construction practice in making those joints was that which is more or less recommended by Professor Davis. As soon as the concrete of a partial lift received its initial set, or as soon as convenient thereafter, workmen removed what laitance they could, and before the concrete was placed for the second lift, a layer of so-called grout, practically cement mortar without much sand in it, was laid. The contractor's equipment was not very big and these sections sometimes required a lot of concrete. At one or two sections which were ruptured by ice thrust we had caused a beautiful polish on the surface of the first lift by the use of cement mortar, almost like a hardwood floor that had been waxed. What happened was that since the partially set concrete was a bit dry, the

*Journal, Amer. Concrete Inst., May-June, 1934, *Proceedings*, Vol. 30, p. 422.

cementing mortar had been robbed of its moisture before the concrete of the second lift was placed upon it. We have used the brushed joint consistently on the invert shoulders of the Winnipeg Aqueduct work which comprised 97 miles of arches running all the way from 6 to 12 ft. wide. The brushed joint was not depended on for water tightness. We used white pine strips for that purpose, half imbedded in the concrete of the invert shoulders; we did get a good water tightness, though not due entirely to the brushing, which was done consistently, as at Professor Davis' suggestion, after the initial set had taken place. To analyze the principles behind these three different joint treatments mentioned by Professor Davis, and to see why they vary in strength—the brushing simply rips up the mortar and leaves it more or less in a state where it may dry and also where it may not be properly filled again with the incoming mass; the use of dry mix also tends to prevent that brushed surface from being filled, because one cannot vibrate the whole joint, one can vibrate the upper mass but not the surface on which the concrete is being placed; washing, with or without air, after the initial set and before the final set, will remove what we used to call laitance in a wet mixed method, corresponding to a skin of inert material; no matter how well placed a mass of concrete may be, one will always find this dead material; the advantage of removing it early as against when the final set has occurred on the surface of the hardened concrete is manifest to experienced engineers. You can attack the surface in one case and you cannot attack it in the other.

John J. Earley: I should like to ask Professor Davis whether the ruptures that occurred, were uniform in the plane of the joints; that is to say, when you looked at the specimen after it had been broken, did the joints just open or did the concrete bridge it in another place?

Professor Davis: Those which were wire brushed consistently broke right along the joint, and in a good many instances the aggregate looked as if it had been polished.

Mr. Earley: The second point: I would not condemn the system of preparing surfaces for bonding by wire brushing, because experience has indicated to us that if the technique be carefully followed, it is one of the best systems for preparing surfaces for bonding.

Professor Davis: We are dealing with mass concrete.

Mr. Earley: I understand that and I understand that it is difficult to draw a picture that would be just exactly parallel in all details between the bonding of concrete in such masses as you use and in such masses as are ordinarily used in building construction, but I cannot reject the idea that the principles must be the same in every case. I recognize

the difficulty of a carefully controlled technique for great masses but our experience indicates that an optimum condition of bonding the structure can be consistently obtained by treatment of the joints by wire bushing; the wire bushing is not the whole story, but the condition of moisture in the concrete at the time of the application of succeeding coats, the condition of the wetness of the surface, because, as I understand it, the principle of bonding is principally to make it grip the matter in question.

Professor Davis: I feel that we must distinguish between the horizontal joints we get in mass concrete and what we get in building work. It would be impractical, of course, to wash horizontal construction joints, in a building job, by the air and water method. It would be very difficult to apply such a method successfully, but I cannot believe that it is possible to secure a more favorable condition for bonding than when the aggregate is perfectly clean; and I do not believe that with wire brushing carried on as it is usually carried on under job conditions, at about the time of final set, we ever obtain the same degree of cleanness of aggregate that can be obtained by the air-water jet prior to the time of final set. Another element helps to explain why we get such distinct differences between the air-water jet applied at early stages and the wire-brushing applied at the time of final set. In traveling all over these surfaces, the brushing method ordinarily tends to disturb the bedment of the coarse aggregates and the cement mortar below.

F. R. McMillan: In support of Mr. Earley's point, I should like to call attention to the work done by the U. S. Shipping Board in the construction of concrete ships during the war. Many will recall that the first ship was constructed as a monolith at great expense; it was decided thereafter to cast the ship in lifts four or five feet high. The wall sections were only four and a half inches thick and as the ship was over four hundred feet long, we had a strip nearly a thousand feet long and only a few inches wide to prepare for receiving the next lift. The surface was cleaned off by wire brushing with results that were one hundred per cent successful. I have seen several of those ships since and have not found any distinct markings between lifts. This indicates the success of the wire brushing. I think Mr. Davis' last remark brings out the probable difference in results between brushing mass concrete which is done with a large broom weighing fifteen to twenty-five pounds, pushed over the surface like a street sweeper, and brushing the small sections of the ship in which hand brushes were used.

The heavy brushing would tend to disturb the particles, thereby weakening the contact between the surfaces.

Mr. Earley: There is another point to which I should like to call attention. It is known by craftsmen that the bond between the newly applied material and the first material is in a thin skin. When the new material is brought into contact with the old material, there is what they call a suction by which the fine particles of the new material are brought down into the old. Now, if that skin be broken in any way immediately after it forms, and it forms almost simultaneously—if that be broken the fissure which exists between the old material and the new is instantly filled, and once that occurs it is impossible ever again to bring those two surfaces into contact; there will always be a space between the two, so if vibration should continue in placing new concrete on old concrete in the zone of the contact, any disturbance of that immediately established bond will tend definitely to weaken that bond, and furthermore that condition itself will set up circumstances in which we might expect joints to permit the passage of water.

A Member: You cannot shake up the first body of concrete, the vibration is all in the second material, it doesn't go down to that skin.

Mr. Earley: If the contact is not disturbed, my argument does not apply; if the contact is disturbed, I am satisfied that it will apply.

Norman G. McDonald (Toronto, Ontario): May I ask the speaker the character of the mortar used, if cement paste was used or if any tests were made with neat cement paste?

Professor Davis: The mortar was of the same richness of mix as the mortar in the concrete of the blocks themselves; it has an eight or nine inch slump. The sand was of the same maximum size and the same gradation as that which entered into the construction of the concrete. That same practice is being followed in the construction of Boulder dam. In that connection we believe that for a number of reasons, the cement mortar, is superior to a neat cement grout.

Mr. McMillan: I do not like to prolong this discussion unduly, but it seems there might be a point made in connection with this mortar supply. I think the purpose of that mortar is often misunderstood. Those who have handled mass concrete with a great proportion of coarse aggregates, realize the need for considerable spading to get a finish on the side against the form. Placing the same mass concrete on a horizontal plane such as the floor of this room, there would be the

same difficulty in getting a smooth face, because of the deficiency of mortar at the plane of contact. In this case there would be no opportunity to spade between the old concrete and the fresh as can be done against the form. By supplying this bed of mortar the deficiency in fine material is overcome. This prevents the continuous band of large openings which would otherwise exist along the surface of contact.

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's corps of Reviewers.

Analysis of thick arch dams including abutment yield

PHILIP CRAVITZ, *Proc. Am. Soc. C. E.*, Jan. 1935, p. 43, Vol. 61, No. 1.
Reviewed by H. J. GILKEY.

Supplies graphical solutions including advances made by Cain, Jakobsen, Vogt and others and including the effect of abutment yield. Illustrative use of the curves is shown.

Tractive resistance as related to roadway surfaces and motor vehicle operation

R. G. PAUSTIAN, *Bulletin No. 119, Iowa Engineering Experiment Station*, Iowa State College, Ames, Iowa, Vol. XXXIII, No. 9, Aug. 1, 1934. Reviewed by H. J. GILKEY.

Data include comparisons of performance on concrete, gravel, bituminous gravel, tar gravel, oil mat, earth, rock asphalt, and other surfaces.

Reinforced glass concrete in theory and practice

H. CRAEMER, *Zement*, Nov. 1, 1934, No. 44, p. 655. Reviewed by INGE LYSE.

Sections of glass are held in position by a light framework of reinforced concrete, the latter for strength and the glass for the passage of light. One of the principal difficulties is in the sealing of the joint between the glass and the frame. The method seems to have found some practical use in Europe as shown by photographs of structures, as well as design diagrams.

Skidding characteristics of automobile tires on roadway surfaces and their relation to highway safety

R. A. MOYER, *Bulletin No. 120, Iowa Engineering Experiment Station*, Iowa State College, Ames, Iowa, Vol. XXXIII, No. 10, Aug. 8, 1934. Reviewed by H. J. GILKEY.

Includes tests on the usual types of highway surfaces and experiments on the effects of dirt, snow, moisture and other factors on such surfaces in their effect on skidding.

Observations on new code of practice for reinforced concrete

C. E. REYNOLDS, *The Structural Engineer*, Vol. XII (New Series), No. 12, Dec. 1934, p. 478-489.
Reviewed by V. P. JENSEN

The Code of Practice of the Building Research Board, in existence for about a year, is discussed in considerable detail. Comparisons are made with other British codes as well as with French, German and American practice. Charts show comparisons of concrete working stresses, proportions of mixes, beam live loads, slab B. M. coefficients, and column bending coefficients.

Norris dam concrete plant

Engineering News-Record, Vol. 113, No. 23, Dec. 6, 1934, p. 726. Reviewed by N. M. NEWMARK.

Describes the central mixing plant for the 1,000,000 cu. yd. of concrete for Norris Dam. A preceding article (*E.N.R.* Sept. 6, 1934, p. 301) describes the aggregate plant. According to the article features of the plant are: simplicity of operation, power-operated proportioning units, and a system whereby the proportion can be changed instantly for a variety of mixes without interruption of flow of material through the plant. With three 3-yd. mixers the rated capacity of the plant is 180 cu. yd. per hour, and the operating crew requires only six men in addition to a foreman and inspector.

A concrete gravity dam for a faulted mountainous area

SAMUEL B. MORRIS and CECIL E. PEARCE, *Engineering News-Record*, Vol. 113, No. 26, Dec. 27, 1934, p. 823. Reviewed by N. M. NEWMARK

The article describes some unusual problems of design of Morris dam on the San Gabriel river in California. The structure, formerly known as the Pine Canyon dam, is a 328-ft. concrete gravity-section dam and the reservoir formed will store 39,300 acre-ft. The design required special provisions for a fault that crossed the site, and for unusually large outlet capacity for streamflow regulation in accordance with complicated water rights. The dam has been designed to resist an earthquake shock with an acceleration of one-tenth gravity.

Frozen cement plaster—new test results from Soviet Russia

W. WICHMANN, *Zement*, Dec. 6, 1934, No. 49, p. 723. Reviewed by INGE LYSE.

Professor Kirejenko at Kiew Institute made studies of proportioning mixes for the mortar, the strength of the mortars after freezing, the methods of plastering at freezing temperatures, and of other related problems. While specifications generally require that no plastering shall be carried out at freezing temperatures the results of the Russian experiments show very successful plastering at low temperatures. The conclusion is reached that cement plastering may well be carried out at freezing temperatures, in fact the results at —20°C. (—4°F.) showed a marked advantage over the results at summer temperatures. A number of diagrams present some of the experimental data obtained.

Determination of the constituents of hardened mortar and concrete

A. STEOPOE, *Zement*, No. 52, Dec. 27, 1934, p. 758. Reviewed by INGE LYSE.

Describes in detail a method of analyzing hardened mortar and concrete for cement content. The author maintains that it is not sufficient to determine the soluble silica content by acids. The residue must be treated with an alkaline solution to give the total silica. To prevent dissolving the silica in the aggregate it is recommended that the cold hydrochloric acid and the cold five per cent solution of carbonic soda be used. It is recommended that separate determinations of the solubilities in acids and alkalines be made on mortars of unknown origin to see if hydraulic admixtures had been used.

Gradation of concrete aggregates for maximum density

GEORG ROTHFUCHS, *Zement*, Jan. 3, 1935, p. 8. Reviewed by INGE LYSE.

The author's experiments showed that when he plotted the amount of material retained on each sieve size against the square root of the opening of each sieve a straight line resulted for the most dense mixture. The formula given for this mixture

is $p = \frac{\sqrt{d} - \sqrt{d_{min}}}{\sqrt{D} - \sqrt{d_{min}}} \cdot 100$, where p is the percentage passing the sieve with diameter d, D is the diameter of the largest particle and d_{min} of the smallest particle. The straight-line relationship provides a simple method of combining different aggregate sizes in such a manner that they give the maximum density.

Special cements

LENNART FORSEN, *Betong*, No. 4, 1934, p. 219. Reviewed by INGE LYSE.

Studies of what types of cements are especially suited for any given purpose have begun in Sweden also and the article by Forsen serves as a summary of the more outstanding results obtained to date in Europe and in the United States on the effect of the chemical composition of the cement upon the physical properties of the concrete. The effect on strength, shrinkage, heat development, workability, permeability, and leaching is considered for different kinds of portland cements. Consideration is also given to the so-called "pansar" cement, a cement having puzzolana as an admixture. Some of the physical properties of the concrete with "pansar" cement are given. The article is well illustrated with diagrams for a number of different conditions.

The great Indian earthquake

Indian Concrete Journal, Oct. 1934, Vol. 8, No. 10. Reviewed by J. C. PEARSON.

A special issue devoted to a description of the effects of the Darjeeling earthquake of Jan. 15, 1934. Scores of excellent photographs are reproduced, showing not only the damage to all types of structures, but many of the peculiar fissures, displacements and blow holes in the earth's crust. Apparently the only undamaged structures in the areas of greatest disturbance were those of reinforced concrete—many small bridges and buildings with light steel framing suffering more or less serious injury. The editorial text gives a general review of earthquake phenomena, with a table of seismic scales of intensity as interpreted by various authorities and a climactic summary of ten lessons from the earthquake. One of the contributed articles is by John Chambers, M. C., A. M. I. C. E., designer of the 300-ft. span reinforced concrete bridge across the Teesta Valley, which withstood the shock without damage.

Standard tests of the commercial cements in Czechoslovakia in 1934

A. GESSNER, *Zement*, No. 51, Dec. 20, 1934, p. 745. Reviewed by INGE LYSE.

Presents the results of tests of all commercial cements manufactured in Czechoslovakia on 14 common portland cements, 1 white and 9 high-early-strength portland cements, 6 blended cements, and 3 slag cements,—a total of 35. Results are given for the fineness of the cements, the time of set, the tensile, compressive and flexural strength, and water requirement for normal consistency. For the common portland cements the compressive strength at two days varied from 1500 to 4400 p.s.i.; at 7 days from 3750 to 7050 p.s.i., and at 28 days from 4750 to 8100 p.s.i. for water-cured mortar specimens. For the high-early-strength portland cements the corresponding variations were from 2850 to 4900 p.s.i. at 2 days, from 4600 to 6450 p.s.i. at 7 days, and from 5950 to 7700 p.s.i. at 28 days. The trend in the strength results as indicated by the average for all the cements of each type is also given for the period 1922 to 1933. The common portland cements show a tendency of increasing strength, while the high-early-strength portland cements show more of a stationary strength tendency.

Twin cableways of long span place concrete at Norris dam

Engineering News-Record, Vol. 113, No. 24, Dec. 13, 1934, p. 747. Reviewed by N. M. NEWMARK.

This article completes the description of the concrete production and placement plant at Norris dam. Previous articles describing the aggregate production and central mixing plants appeared in *E. N. R.* Sept. 6, 1934 and Dec. 6, 1934. The article describes the two traveling cableway units of 1928-ft. span between towers operating on tracks set high on the sides of the river valley. The cableways handle not only all the concrete, but also forms, steel, and other supplies and equipment. Each of the units has a rated capacity of 18 tons, and operating in unison have already handled 40-ton sections of the steel penstocks. Features of the system are: full electric operation, telephone signalling, and the use of specially designed buckets of 6-yd. capacity. The buckets are discharged by compressed air. Under favorable conditions it has been possible to maintain a rate of placing of 20 buckets per hour, and more than 2000 cu. yd. of concrete has been deposited by one cableway in a working day of 22 hours.

Investigations on a portion of the quaternary system $CaO - Al_2O_3 - SiO_2 - Fe_2O_3$: *The quaternary system* $CaO - 2CaO.SiO_2 - 5CaO.3Al_2O_3 - 4CaO.Al_2O_3.Fe_2O_3$.

F. M. LEA and T. W. PARKER. *Philosophical Transactions of the Royal Society of London*, Series A—Mathematical and Physical Sciences No. 731, Vol. 234, pp. 1-41, Plate 1, Nov. 9, 1934.
Reviewed by ROY N. YOUNG.

Phase relationships as determined for the Quaternary System $CaO - 2CaO.SiO_2 - 5CaO.3Al_2O_3 - 4CaO.Al_2O_3.Fe_2O_3$ are reported together with theoretical discussions, methods and complete experimental data. This system is the region in the major system $CaO - Al_2O_3 - SiO_2 - Fe_2O_3$ in which portland cement occurs. The authors have relied upon the previous works of Rankin and Wright; Hansen, Brownmiller and Bogue; and Hansen and Bogue for phase rule relationships of binary and ternary systems involved in the major quaternary system but to complete the information needed to start with the quaternary system it was necessary to determine the phase relationships of the systems $C_2S - C_4AF$, $CaO - C_2S - C_4AF$ and $C_2S - C_5A_3 - C_4AF$. No compounds were found in the quaternary system that have not been previously reported in the studies of the smaller systems by the authors mentioned.

Earthquake resistance of structures

C. W. HAMANN, *The Structural Engineer*, Vol. XII (New Series), No. 12, Dec. 1934, p. 492-507.
Reviewed by V. P. JENSEN.

Faulting, as contrasted with volcanic earthquakes, is recognized as producing vibrations of greatest significance in structural design. Waves are classified as longitudinal primary depth waves, secondary lateral depth waves, and surface waves. Assumption of simple harmonic motion leads to accelerations when amplitude and period are observed.

For reinforced concrete construction the importance of adequate stirrups in beams and of spiral reinforcement or ties in columns is pointed out. Beams and columns must be well tied together. Ample reinforcement for negative bending is essential. Danger of fire which accompanies earthquakes leads to a recommendation of at least 2 in. of concrete fireproofing on all steelwork, the fireproofing to be held by spiral reinforcement. Foundations should be deep and footings connected.

Effects of shock are greatest—(1) at, or adjacent to an actual fault line; (2) at the bottom or top of cliffs; (3) where two different types of rock are adjacent; (4) in made ground.

Common design procedure is outlined. A reference list is appended.

Construction of a reinforced concrete double hangar at the aerodrome of Metz-Frescaty.

BERNARD LAFFAILLE, *Le Genie Civil*, Vol. CV, No. 19, p. 425 to 429. Reviewed by R. L. BERTIN.

Following a competition on a project instituted by the author, the Air Ministry of France awarded to Rouzand & Sons the execution of the above named structure, consisting of two adjacent sections, each 50 m. square, separated by a beam 4 m. wide. The clearance under the tie beam is 10.50 m. The construction is composed of vaulted ceilings, having spans of 50 m. which rest on the exterior on a line of posts and in the center on a common beam. The outer posts and center beam were first installed, then the vault and tie beams were built progressively, using a traveling form work actuated by jacks.

The novel features of this structure, claimed by the author, are two fold: (1) Use was made of the mathematical properties of thin shells curved in one direction relative to equalizing the stresses due to dissymetrical wind forces. (2) Precast specially shaped posts for the outside supports and rear wall were used in preference to cast-in-place units. The vault shell is 6 cm. thick and the ties are spaced 7.80 m. Two stiffening ribs are provided between the ties to prevent the buckling of the thin slab.

Bleeding of Cements

LEVI S. BROWN, *Industrial and Engineering Chemistry*, Jan., 1935, V. 27, No. 1, p. 97-102. Reviewed by ROY N. YOUNG.

The phenomenon termed "bleeding" of cement has been investigated by examinations of cement pastes and hardened concrete using several commercial cements. The bleeding tendency was determined by dispersing cement thoroughly in water in a test tube and then measuring the amount of supernatant liquid after allowing the paste to stand for two hours. The water-cement ratios were varied from .7 to 1. Concretes were made with corresponding cements and water-cement ratios using a proportion of 1:6 by weight. From the hardened concrete thin sections were made at contacts between the mortar and large aggregate for petrographical examination. A comparison of the results of the bleeding tests and the photomicrographs of the thin sections shows a relationship between the amount of bleeding and the presence of channels under the aggregate in the hardened concrete which indicates that bleeding is at least one of the causes for weaker bond on the under side of large pieces of aggregate in concrete than at their top surface as has been observed by other investigators. This tendency toward the occurrence of channels or thin openings under the larger pieces of aggregate is shown to be decreased by lower water-cement ratios, higher cement fineness and the use of a dispersing agent called T. D. A.

Observations on a general theory of the shrinkage of hydraulic cements

M. PIGEAUD (Inspecteur General des Ponts et Chaussees) *Les Annales des Ponts et Chaussees*, 1934, IV, July-Aug. Reviewed by B. MOREELL.

M. Pigeaud reviews Freyssinet's corpuscular theory of the composition of cements and points out certain anomalies which are not explained by Freyssinet. He develops a modification of Freyssinet's theories based, also, upon the fundamental conception that volume changes in cements are largely controlled by the magnitude of the surface tension of the water entrained in the capillary channels. He states that not only does the hygrometric state of the surrounding air effect the rate of dessication of the cement, but that the temperature and pressure of that air are important

factors. The temperature is of particular importance in that it has a marked effect on the magnitude of the surface tension of the couple air-water.

An important conclusion is that as long as the cement corpuscles have not reached their final stage of evolution (and this, in the average cement, requires several years), there are, in reality, no mechanical or thermal constants. The coefficient of expansion, modulus of elasticity, and Poisson's ratio are constantly changing and one can accurately speak only of instantaneous values.

A review of M. Freyssinet's corpuscular theory of cements was published in the November-December 1933 JOURNAL of the Institute. Those who are interested in this subject will profit by a careful study of M. Pigeaud's paper.

Concrete columns encased in steel shells proposed

WILLIAM S. LOHR, *Engineering News-Record*, Vol. 113, No. 24, Dec. 13, 1934, p. 760.
Reviewed by N. M. NEWMARK.

A type of concrete column is described that consists essentially of a concrete shaft encased in a steel tube shorter than the shaft with some coating material between the concrete and the steel to prevent, so far as possible, any adhesion between the concrete core and the tube. It is this attempt to relieve the encasing tube of longitudinal compressive stress which constitutes the essential difference between the encased column and the Lally column. A patent has been granted on the basic principles involved but it is not clear that these principles are different from those involved in a spirally reinforced concrete column without vertical reinforcement.

The inventors claim certain advantages of encased columns over other types of concrete columns, and the author states that the preliminary studies lead to the conclusion that it is possible to save some 40 per cent of the cost of columns in reinforced concrete buildings by the use of encased columns.

It is admitted that various objections and criticisms of the proposed type of column can be properly answered only after exhaustive and carefully made tests. Such objections concern the possibilities of flexural weakness of the columns both as a whole and also at the joint at each floor level, and the probability of excessive longitudinal deformation. A further objection not mentioned in the paper concerns the bonding of the fireproofing to the steel shell.

Use of high elastic limit steel as reinforcement for concrete

H. J. GILKEY and G. C. ERNST (Report of Project Committee), 14th Annual Proceedings, the Highway Research Board. Reviewed by the HIGHWAY RESEARCH BOARD.

By the analysis of the problem presented in this preliminary report and the accompanying review of the existing literature pertaining to the subject, the Committee has been able to formulate the known facts concerning the use of high elastic limit steel as concrete reinforcement and to point out the factors upon which further research is needed.

The lower ductility of high elastic limit steel bars does not constitute a valid objection to their use. Bars rolled from steel railway rails exhibit satisfactory uniformity. As longitudinal reinforcement in columns this grade of steel supplies additional strength. Fatigue and impact loading and volumetric change in concrete do not present any special problem.

Among the factors needing research are: welding, bond strength, deflection of flexural members, affects of tensile cracks, identification of grades and severe impact.

At the same working stresses high elastic limit steel can be safely and satisfactorily used interchangeably with softer material but if economy is to be achieved

advantage must be taken of the higher possible working stresses for both steel and concrete. Research is needed to clear up questions bearing upon this point. The report analyzes the possibilities of saving through the use of higher working stresses and alterations in design procedure and applies them to the redesign of a deck girder highway bridge. A considerable saving in material is shown to be possible.

Investigation of commercial masonry cements

JESSE S. ROGERS and RAYMOND L. BLAINE, *Research Paper* RP746, National Bureau of Standards, Vol. 13, Dec. 1934. AUTHOR'S ABSTRACT.

Forty-one commercial masonry cements were studied with respect to chemical composition, fineness, weight per unit volume, volumetric flow of the neat pastes, and bulk specific gravity, while mortars made from those cements were studied with respect to resistance to deformation, water-retaining capacity, volume, yield, linear changes, compressive and transverse srtength, efflorescence, durability when subjected to cycles of freezing and thawing, and absorption.

It was found that the cements could be classified as hydraulic limes, hydrated limes, natural cements, blast-furnace-slag cements containing various additions, several cements whose composition could not be positively determined, or portland cements with and without admixtures, the quantities of which varied from small amounts to amounts larger than the quantity of portland cement. About half of those studied contained water-repellent additions.

The physical properties of the mortars made from the cements also varied over a wide range. For example, the weight per cu. ft. of cements varied from 39.7 to 89.9 lb. The compressive strength of the mortars when tested at 28 days ranged from 50 to 3,650 p.s.i. The addition of water-repellent material strikingly affected the properties of mortars made from cements to which such additions had been made. The workability particularly seemed to be increased, due to the incorporation of air in the mortar during mixing, brought about by the water-repellent additions acting as emulsifying agents.

A discussion of the essentials of a specification for masonry cement is presented.

Maintenance and care of concrete roads

FEUCHTINGER, *Beton u. Eisen*, Vol. 33, No. 12, p. 190, June 20, 1934.
Reviewed by A. A. BRIELMAIER.

A consideration of the present status of concrete roads in Germany. German concrete roads may be divided into three groups: those dating as far back as 1888; those built between 1925 and 1930; and those constructed since 1930. The first group withstood the pre-war traffic but is failing for lack of maintenance. The second serves as experimental studies on which the third group is based.

A comparison of concrete roads of various countries requires consideration of the influence of climate, traffic and materials. The roads here discussed are used by iron-wheeled wagons and horses as well as automobiles. These various factors prevent the arbitrary application of experience gained under a different set of conditions.

The following points are brought up in some detail. Joints are to be filled with an elastic material, and the filling inspected and supplemented at the beginning of summer and winter. At the present time, cracks cannot be eliminated but their size and number can be limited. Hair-cracks are not dangerous unless they denote the separation of the surface and the base. There should not be much variation in cement content of the surface and base. The use of high strength and ordinary cement in the same slab is not advisable. Old macadam that is to be covered with a concrete slab should be brought to the shape of the proposed concrete; all uneveness

should be removed. Attention to underdrainage and foundations is of primary importance. Provisions should be made to prevent cracks at man-holes and other breaks in the concrete. Experience and maintenance have indicated the superiority of a stiff mix pneumatically tamped, over a wet mix. Longitudinal joints are advisable in slabs wider than 18 ft. Transverse joints may be placed at intervals of 30 ft. and greater.

It is becoming customary for the contractor to assume responsibility for road maintenance for perhaps the first five years, without special compensation; and for a further five or ten years with compensation. In some cases the maintenance costs for the first five years vary from $0.001 to $0.0015 per sq. ft. per year; for the second five year period from $0.002 to $0.005 per sq. ft. per year.

New Traneberg bridge in Stockholm

JUL FRANK, *Zeitschrift des Osterr.-Ingenieur—und Architekten-Vereines*, Heft 43-44, Nov. 2, 1934, p. 251. Reviewed by INGE LYSE.

This, the largest span arch bridge in the world, was opened for traffic by the Swedish Crown Prince, Aug. 31, 1934. The bridge consists of a main arch span of 594 ft., (the George Westinghouse Bridge has a span of 460 ft.) and a number of approaches, giving it a total length of 1525 ft. It connects the city of Stockholm with one of its suburbs. Competitive designs showing estimated cost of about 1.2 million dollars for reinforced concrete construction against 1.4 million for a steel structure, the reinforced concrete design was adopted. The main section consists of two independent hollow type arch ribs of 30 ft. width and depth varying from 16.5 ft. at the abutment to 10 ft. at the crown. The clear distance between these ribs is 20 ft. The distance between the roadway railings is 90 ft. within which are provided areas for a double track interurban railroad, 40 ft. roadway, and bicycle lanes and pedestrian walks.

The bridge is designed for very heavy loads, the roadway for four parallel rows of continuous double-axis trucks of 15 ton weight and the lanes and sidewalks for 27.5 per sq. ft. uniform load. The railroad is designed for a four-axis locomotive of 68 tons and three four-axes cars of 40 tons each on each track. Allowance has been made for dynamic effects.

Special high grade portland cement and high grade aggregate were used in the concrete. The permitted maximum stress in the concrete was generally 1420 p.s.i. with a maximum of 1700 p.s.i. for the crown section of the arch ribs. The reinforcing steel had a minimum yield-point stress of 42,500 p.s.i. Steel arches supported the concrete forms for the ribs, these arches being moved from one rib to the other. The concrete was delivered by a central mixing company to insure the highest degree of uniformity in quality. In part of the bridge concrete pumps, used for the first time in Sweden, pumped concrete successfully a distance of 660 ft. and height of 130 ft.

The Freyssinet method of introducing initial moment in the arch ribs was used. A total of 25,000 cu. yd. of concrete, 1300 tons of reinforcing steel, and 1300 tons of structural steel beams were used in the bridge. The principal part of the construction was completed in 15 months.

Electrolysis and corrosion damage warehouse

DON HULL MCCREERY, *Civil Engineering*, Vol. 4, No. 11, Nov. 1934, p. 572. Reviewed by J. R. SHANK.

A two story warehouse, concrete flat slab construction, in Los Angeles, used for storage and curing of hides, built in 1925, was found to be in a seriously damaged

condition in 1932 because of salt drip from the hides. Fresh hides with layers of salt between are stacked on the floor in piles for 30 days or longer for curing. The reinforcing steel of both slabs and columns was attacked most severely by the salt drip. Only in scattered instances at construction joints and expansion cracks could any decomposition of the concrete be detected. In two years after construction salt water literally poured through the slab construction joints. At this time an asphalt mastic covering was placed on the floor and the columns were painted with a black coal tar paint but neither of these stopped the damaging action.

The corrosive action on the reinforcing steel seemed to be of two kinds, one a corrosion oxidation with expansive action which loosened or spalled off the concrete and the other an electrolytic chlorine or hydrochloric acid action which did not act to loosen the concrete though the damage to the steel was greater.

The damage was largely confined to the walls and columns of the basement and first floor and the first floor slab. The spiral steel of the lower 5 ft. of the columns was corroded so badly as to be useless, and examples were found where longitudinal steel was more than half eaten away. This was aggravated by the condition that in 30 per cent of the columns the steel was 1¾ in. off center. Every linear foot of the basement wall was affected to a height of from 2 to 8 ft. Every bar was affected and in some cases 95 per cent of the bar was eaten away. Out of 750 lin. ft. of negative steel cut from the slabs only 40 lin. ft were fit for use and this condition obtained on three-fourths of all of the negative areas. Electric conduits were so badly corroded that in many cases they were filled with salt water.

The concrete of the columns at all damaged areas was chipped off so as to remove all of the spiral steel and uncover and expose all sides of the longitudinal steel. Damaged longitudinal steel was cut out and new steel was arc welded into place. New spiralling was wrapped around and the column was built out with gunite 1:4 to its original size. This same general procedure was used on the walls and slabs excepting that 1:3 gunite was used on the slabs.

A combination of asphalt enamel and soft asphalt enamel reinforced with burlap was used for water-proofing the slabs. The concrete was primed with an emulsified asphalt or cut-back asphalt primer.

Design of flat slabs

M. STEUERMANN, *Beton u. Eisen*, Vol. 33, No. 17, p. 273. Reviewed by A. A. BRIELMAIER.

The author had charge of the design of several million square yards of flat slabs constructed by the Refrigerating and Packing Plant Trust in Russia. The article is taken from the studies made to determine the methods to be used in design. It deals with the results obtained by the methods of Marcus and Lewe based on theory, and American practice, based on tests.

For a theoretical analysis, the structure can be viewed as a system composed of two continuous frames transverse to each other. The moments in the horizontal members, which are the flat slabs, will not be resisted uniformly across the width of these members, but according to the dimensions and type of the salb. It is to be emphasized that the total moment must be provided for.

A two-way flat slab panel (16 ft. by 16 ft.; design load of 245 lb. per sq. ft.) is analyzed for moments by the three methods and the moments compared. There is no agreement for the values found for the moments in the center and column strips. The total moments (sum of positive and negative) across the panel agree within 4 per cent for the Marcus and Lewe analysis. The American totals are 70 per cent of the others. In considering the low moments obtained in American

practice, the parts played by "arch action" and tension in the concrete have been mentioned. The author's studies have indicated that arch action may account for one to two per cent of the total moment. The steel totals (A_s) for the slab width with a thickness of slab of 6.3 in., are in the same relation to each other as the above-mentioned moments. If, however, the values found according to the theories of Marcus and Lewe are based on the concrete taking tension, they vary only a few per cent from the American steel areas.

As a result of the studies, the flat slabs were analysed according to Marcus and the sections determined according to a method of the author.

The author's views of American flat slab practice mention the following points:

The total moments vary greatly from those found by theoretical analysis.

Formulas for slab thickness agree satisfactorily with corresponding European formulas.

There is a slighting of the influence of the columns in resisting bending of the slab.

An uneconomical design results unless the panels are square or nearly square.

The steel areas agree with those found by theoretical methods of analysis, if the computations of the latter are based on the assumption that the concrete takes tension.

The aerodynamic wind tunnel of Chalais-Meudon (Seine-et-Oise)

G. LEMAREC, *Le Genie Civil*, Volume CV, No. 18, p. 401 to 406. Reviewed by R. L. BERTIN.

The author describes the design and construction of the above named structure which is a testing station for the study of full size aeroplanes under actual wind conditions.

The structure, which is entirely of reinforced concrete, consists of a horizontal elliptical funnel called collector which serves to gather, filter and orient the outside air. It connects with the experimental chamber in which are set the aeroplanes to be tested. Opposite to the collector, a diffusion chamber in the form of an elliptical pipe, gradually increasing in size connects the experimental chamber with the suction space, the outer wall of which is provided with 6 large exhaust fans which create air currents capable of a speed of 180 km. per hour.

The collector is 15.36 m. long, elliptical in cross section, the major axes at the inlet and outlet being 24.8 m. and 16.8 m. respectively and the corresponding minor axes 16 m. and 8 m. It is constructed of a 7 cm. reinforced concrete shell stiffened with ribs on the extrados. It bears on a reinforced concrete frame at the inlet and the wall of the test chamber at the outlet.

The test chamber is approximately 20 m. wide and 24 m. high, covered with a curved saw tooth roof.

The diffusing chamber consists of an elliptical reinforced concrete tube 38 m. long, spanning a clear distance of 34 m. between the experimental chamber and the suction space. The walls are 7 cm. thick stiffened with ribs 16 x 60 cm., spaced approximately 3.60 m. on centers placed on the extrados.

The horizontal axes vary from 18 to 23 metres and the corresponding vertical axes from 10 to 15 metres.

The suction space is 29.50 metres long and varies in width from 36 to 42.5 meters. This space is covered with an arched roof varying from 22.50 to 26 m. high. The arch consists of a thin reinforced concrete shell stiffened with ribs, the thrust being resisted by means of inclined struts from the spring line 14 m. above the floor to the ground.

The principles involved in the design of the elliptical tubes are given, and many of the details described and shown cannot be reviewed advantageously without the numerous illustrations.

The structure, begun July 1, 1932 and completed Oct. 1, 1934, is a remarkable example of the adaptibility and possibilities of reinforced concrete in the solution of the most varied and complex problems.

Wheel—load distribution on concrete pavements

R. D. BRADBURY, 14th Annual Meeting, Highway Research Board, Dec. 1934.
Reviewed by R. W. CRUM.

In the Westergaard analysis of stresses in concrete road slabs the radius of load distribution a is an important factor. Since the computed stress in the slab may have a very appreciable range of variation depending upon the value of a, it is necessary that this factor for various wheel loads as affected by different types and arrangements of tires be determined before rational stress analysis can be applied.

The size of contact area per tire largely determines the degree of load distribution effect. In the case of a wheel load applied through the medium of dual tires the spacing of the tires may also have a pronounced influence upon the value of the effective radius of distribution.

The objective of this analysis is the evaluation of proper allowable or working distribution factors suitable for general use in stress computations for given wheel loads with the tire equipment most likely to be used in connection with that loading.

The load distribution factor for solid and cushion tires must be obtained experimentally, but for pneumatic tires the load-inflation quotient closely approximates the contact area. This discussion is confined to pneumatic tires.

Where dual tires are involved an equivalent radius of distribution for the wheel load is readily obtained from a proper combination of the separate slab stresses produced by each tire.

By charting values of the distribution radius it has been found possible to express by an empirical formula the virtual radius a in terms of the wheel load for each load position and tire type.

Examination of the recommendations of the Tire and Rim Association show that for wheel loads in excess of 3000 lb. the cost per wheel is less for duals than with single tires, which largely accounts for the prevalence of dual tires on comparatively light trucks and busses.

Analysis of relative costs indicates that one may assume for the purpose of this analysis that the most probable types of tire equipment, exclusive of passenger car balloons, may be: high pressure single for wheel loads up to 3000 lb., high pressure duals for 3000 to 8000 lb., and balloon duals for wheel loads over 8000 lb.

It is assumed that the great majority of operators will adhere rather closely to the load inflation recommendations of the makers.

Consistent working values for the equivalent radius of load distribution for single tire contacts may be computed from $a' = 0.250 \sqrt[3]{P}$ for circular distribution and $a' = 0.354 \sqrt[3]{P}$ for semi-circular distribution. P is the load on the tire.

Representative and consistent values of the distribution radius for dual tires may be computed from the following formulas: for corner loading $a_c = {}_{0.42}\sqrt[3]{W}$, for interior loading $a_i = 0.073 \sqrt{W}$, for transverse edge loading $a_e = 0.17\ W^{0.42}$, for longitudinal edge loading $a_e = 0.16\ W^{0.45}$. W is the total load on the wheel.

Studies of vibrated concrete made of "thinned" cements for the Sansanding Dam, (Niger river—Africa)

E. MARCOTTE (Chief of the Physical and Mechanical Section of the Laboratory of the Ecole des Ponts et Chaussees), *Les Annales des Ponts et Chaussees*, Sept.-Oct., 1934. Reviewed by B. MOREELL.

Because of large quantities of materials required for this important work, the transportation difficulties, and the desirability of reducing to a minimum the heat of hydration, special studies were made to determine the optimum composition and methods of placing the concrete.

Local aggregates were satisfactory; but it was impracticable to install a cement mill on the site. To reduce the quantity of cement to be transported, it was proposed to ship clinker in bulk from France and to grind it at the site with a "thinner" of local sand. This product has been used in the United States under the name of "sand-cement."

Preliminary tests indicated that the sand from the river Niger was the best suited for this purpose, and that the optimum mixture consisted of equal weights of sand and clinker. To determine the optimum concrete proportions, a series of strength and permeability tests were made.

As the climate at the site is tropical the tests were designed to furnish information on the effects of high temperatures and humidities. Special closets were devised to permit fabrication and storage of specimens under variable temperatures and humidities. Recording indicators were used to determine the time of set and the time-temperature curves during setting. The tests furnished information on the following points: (1) Comparative strengths of vibrated and non-vibrated specimens; (2) Comparative permeabilities of vibrated and non-vibrated specimens; (3) Effects of variations of curing temperature; (4) Results obtained with "thinned" cement.

The tests are discussed in considerable detail. Some of the conclusions are summarized as follows:

Vibration permitted the use of considerably less water, with increased strengths. Two minutes on a vibrating table produced strengths of about 70 per cent of those obtained by vibrating 15 minutes with an internal vibrator and a platform vibrator.

The permeability was greatly reduced by vibration; a non-vibrated concrete containing 200 kg. of cement per cubic meter of concrete showed approximately 500 times more leakage than a vibrated concrete containing 175 kg. of cement per cubic meter, with a much lower water cement ratio. Concrete specimens aged at 40°C. in a saturated atmosphere (simulating conditions at the site) were stronger than specimens cured under normal conditions (15° to 18°C.). At 7 days the difference was 18 per cent for specimens made with standard cement and 26 per cent with "thinned" cement, while at 28 days the differences had diminished to 2 per cent and 5 per cent respectively.

For equal "cement factors," concretes made with "thinned" cement showed a strength 20 per cent greater than ordinary cement concrete even at early ages, and, on account of the better gradation of cement and sand particles, the former had a better workability and responded better to vibration.

Concrete of "thinned" cement was considerably less permeable than that of standard cement with the same cement factor.

It should be noted that the characteristics of the "thinning material" are very important. For example, when ground bauxite was used as an admixture, in lieu of sand, the compressive strength of the concrete *decreased* with age. It appeared that some unfavorable chemical action was taking place between the fine particles of bauxite and the cement.

The time of set was much longer for sand-cement than for standard cement, but was decreased by increasing the temperature of the air.

It is concluded that where a suitable "thinning material" is available, it might be advantageous to use "thinned-cement."

This article is the result of a partial study of the question; more experiments are necessary to elucidate all of the useful points. (Note by translator: It should be noted that the article does not discuss the important matters of heat of hydration and volume change of "thinned" cements.)

Effect of calcium chloride on portland cements and concretes

PAUL RAPP, 14th Annual Proceedings, Highway Research Board.
Reviewed by the HIGHWAY RESEARCH BOARD.

To study the effect of the addition of calcium chloride on portland cements of widely varying composition, and on portland cement concretes, a carefully controlled laboratory investigation was conducted. Measurements of heats of hydration, setting times, consistencies, and the strengths at various ages under several curing temperatures were made.

Eight commercial portland cements, one high-early strength, and two white portland cements were studied. Certain heat and strength studies were also made on 60 experimental cements having a wide range in composition and on four commercial portland cements.

The heat of hydration was measured directly and from these data, the heat evolved by each cement was calculated. The contributions of the individual compounds to the heat of hardening, for the 60 experimental cements, were calculated by the method of least squares from the percentages of the compounds in the cements. Heats evolved during the first 24 hours of hydration were measured for 11 commercial cements without calcium chloride and with the addition of 0.5, 1, 1.5, and 2 per cent of anhydrous calcium chloride.

In the strength tests on mortars and concretes, all factors were controlled as far as possible so that the amount of calcium chloride was the main variable. For the experimental cements, 1-in. mortar cubes for three ages and for 0, 1, and 2 per cent added calcium chloride were made. Standard evaluation tests for the commercial cements were made, except that 2-in. cubes were substituted for briquettes. In these tests, 0, 0.5, 1, 1.5, and 2 per cent of calcium chloride were added to the gaging water. Concrete specimens were 6 by 12-in. cylinders, 1:2:4 mix, with 0, 1, 1.5, and 2.25 per cent of anhydrous calcium chloride added. The effect of three curing temperatures on strength was studied by plastic mortar tests. The three conditions were: (1) at 70°F. for 90 days; (2) at 40°F. for 28 days then 70°F.; (3) at 90°F. for 28 days, then 70°F. All storage after the first day was in water.

The addition of calcium chloride appears to increase somewhat the heat contributed at the end of 24 hours by dicalcium silicate and tetracalcium alumino-ferrite and to decrease the heat contributed by tricalcium aluminate. Calcium chloride has but little effect on the heat contributed by tricalcium silicate. Calcium chloride increases the rate at which heat is evolved from all cements. For experimental cements which developed 30 to 40 calories per gram during the first 24 hours, the addition of one per cent of anhydrous calcium chloride increased the heat 7 calories per gram. Cements which liberated greater amounts of heat had a smaller increase in heat due to calcium chloride, until the group liberating from 70 to 80 calories per gram showed a decrease of 2 calories per gram when calcium chloride was added.

In the study of the 60 experimental cements at ages up to 28 days, it was found, as by other investigators, that the compound composition greatly influences the

strength. Calcium chloride increased the strengths of these cements in mortars, especially marked when the composition was within the range encountered in commercial cements. The strength contributed by the dicalcium and tricalcium silicates in cement was increased and that contributed by tricalcium aluminate was decreased by the addition of calcium chloride.

For the 8 standard, 2 white and 1 high-early strength cements, the setting times at 70°F. were decreased by the addition of increasing amounts of calcium chloride. The strength of all the cements was increased by calcium chloride at all ages up to one year, which was the maximum age of test. Early strengths were greatly increased and one-year strengths appreciably. The flow or workability of the concrete was increased by the addition of commercial calcium chloride in amounts up to 3 per cent.

For the three curing temperatures studied, the optimum amounts of commercial calcium chloride which should be used were found to be:

(1) At 40°F., 2 per cent for all cements.

(2) At 70°F., 2 per cent for standard portland and white cements and 1.5 per cent for high early strength cement.

(3) At 90°F., 1.5 per cent for all the cements.

Concrete — Yesterday, Today, Tomorrow*

BY EDWARD J. MEHREN†
MEMBER AMERICAN CONCRETE INSTITUTE

True to their trust, the directors of the American Concrete Institute asked themselves recently if the concrete art was in tune with the times; whether in a period of speedy change it was alert and virile. They suggested an assay of the present position of the art and an inquiry into its near future. Where do we stand today? Whither are we going? Such are the questions they want to have answered.

THE PATH WE HAVE TRAVELED

But before we attempt to answer them it will be helpful if we ask another question: By what path have we arrived where we are today? A look backward will better enable us to appraise our present position, and to project the curve of the future.

An historical inquiry might well go back to ancient days, for our material comes of old and honorable lineage. We might very well refer to the work of the Romans. But, first, you know that story very well, and, second, we can get more light on our problems, in the short time available for this address, by concentrating on the period of which we are a part. For while much was accomplished prior to the turn of the century, the year 1900, in the United States at least, marked a great upward turn in our art. It was then that reinforced concrete, known at that time for 33 years, got its firm foothold in the United States.

But there is another reason for concentrating on these years since the century began. We celebrate today the thirtieth anniversary of the founding of this Institute. What could be more appropriate, then, than a review of what has happened since the hopeful pioneers of this body set it forward on its great mission?

But even with this limitation it is difficult to treat our subject adequately in the short time at our disposal. Our material, concrete, is very versatile. We dare not follow it into all its uses; we cannot make a full appraisal. Possibly, however, we may get a passable

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understanding of its development if we confine our inquiry to these heads—first, concrete itself; second, cement; third, design.

I conceive that in the year 1900 there was working in each of these branches of our art, a small group of valiant men whose characteristics were ability, foresight and courage. Ability they had, for theirs became the task of adapting a new material to the exactions of engineering science. Foresight was theirs, for only a vision of the future could have justified the energies they expended; courage, for the task was difficult, the discouragements many. Yet see what happened. Recruits of outstanding ability rushed to their standards—scientists, engineers, architects, constructors, inventors of machinery. Soon each little group became a band; the band swelled to a regiment; the regiment, to a beneficient army; until today those armies place in our hands rich treasures of experiment, knowledge and experience, which enable us to use this versatile material soundly, efficiently, beautifully. That which we do, therefore, in our brief historical review is to record the progress of these three armies and to appraise the richness of the knowledge they have developed and which they now offer the practitioner.

What, then, is the contribution of the army of the left, that which dealt with concrete itself?

CONCRETE—THE MATERIAL

The year 1900 found us in the age of the very dry mix, so dry that without thorough, careful and heavy tamping the material was a honeycombed, low-strength mass. In trying to remedy its defects practice swung in time to the other extreme. The ultra-dry mix gave way to slop. We wanted something that would flow, or run—and we got it. On mass jobs men waded calf-deep in soup. Fiendish planning could not have done worse.

But this could not go on. The army of the left had begun to deploy i ts forces. Its keen men in field and laboratory had seen a light. They had begun to ask questions. What was the effect of varying the quantity of water? What had proportioning to do with strength, impermeability, weather resistance? What was the relation between workability and an otherwise optimum mix?

The pursuit was speedy and relentless. In a few years there emanated Abrams' water-cement ratio law and Talbot and Richart's void-cement ratio method. The first great step had been taken. Concrete had ceased to be a chance agglomeration. It had laws; it could be designed for a given strength; it had become a rationalized engineering material.

CONCRETE STEPS INTO THE FIELD

And still the army marched on. The scientists had told us how to get what we ought to have, but each engineer and contractor was a law unto himself. Would he accept the new learning?

Here opportunely a new band joined the advancing throng. Concrete was making strides as a paving material, a severe service where only high quality was equal to the task. Further, uniformity was essential, for poor batches here and there were as ruinous as if the whole had failed. The new learning pointed to the solution and the highway engineers seized it eagerly. The new concrete—the rational, designed concrete—was invited to step out of the laboratory and play its part in practice. The highway engineer had joined the army of the left.

CONTROL DEVICES

However, a new technique of construction was needed if the revolution in the art was to succeed. There had to be simple means of securing from batch to batch rigid adherence to the proportions and water-cement ratio that might be determined. The Iowa Highway Department joined the army of innovators and specified that all materials, including water, should be weighed. The last phase of the revolution was under way.

The response was rapid. Weighing became well nigh universal in highway work and spread to mass and other structures.

Quickly there came belt conveying and ready-mix plants, both requiring quality control, and, most recently, the pump for concrete—a placement device that seems endowed with almost intelligence in refusing an improper mix.

Then came another offering, but with a different objective, namely, that of securing very high strengths in practice. I refer, of course, to vibration—doubtless the greatest step forward in concrete making since the discovery of the water-cement ratio law.

If you would know whither the path over which the progress of that army has brought us and would measure the art of concrete making today against that of the year 1900, I would hold before you, first, the periods of the dry-mix and the deluge, when performance was measured solely by output, and second, the design and control technique on the Boulder and the Norris dams and on the structures being built under the Corps of Engineers of the Army, where with output vastly greater, quality and uniformity are achieved in the highest degree. In these outstanding works we find the new learning at its

best in working clothes, the full maturity of present-day knowledge of concrete, the fruition of the work of the army of the left, the army of concrete itself.

PROGRESS IN CEMENT

What meanwhile was happening in the army of the center, which concerned itself with cement?

From 1900 to 1920 that group had brought us steady improvement in quality—greater uniformity, finer grinding, higher strength. But the explanation of various qualities—strength, rate of hydration, volume change—was far from satisfactory or complete. And there was hardly a suspicion that the rate of heat generation during hydration had any practical influence.

Here again the highway engineers played an important role. They had begun the regular testing of cement. As a result, variations in early strength were discovered. Why? why? was the insistent question, all the more important because early strength determined the time for putting a new pavement into service.

Here, too, the design and control methods discovered by the other army played an important part. With every element under control, the variation in cement could be isolated. Thus did progress in concrete-making spur inquiry into the cementing medium.

Meanwhile research in cement had long been under discussion by the cement manufacturers. Their cooperative research in concrete had proved highly fruitful, and helped to demonstrate the need for a more scientific understanding of cement. As a result, the Portland Cement Association Fellowship for the study of cement composition was taken in hand in 1924 at the Bureau of Standards.

It was none too soon. Practice had already begun to put pressure on knowledge, and other pressure from practice was in the offing, though none foresaw it in 1924.

CHANGE IN COMPOSITION

During the same period methods of construction had sharply increased the yardage that could be placed in mass structures in a given time. The rate of heat evolution, negligible with lower speed of placement, became important. Different cements, too, evolved heat at different rates. Why? Happily before the question became acute the Cement Fellowship, under Dr. Bogue, had laid the foundation for the answer. It had identified tetracalcium alumino-ferrite, a discovery which provided the basis for the quantitative determination of the major compounds. Building upon this foundation it had studied the

properties, including heat evolution, of the four major constituents. The learning was at hand to change cement to meet the new needs.

Starting from the base furnished by the Fellowship findings, Hubert Woods of the Riverside Portland Cement Co., had carried out extensive investigations on the basis of which the company produced cement for the Rodriguez dam, in Mexico, through which, for the first time, heat evolution was controlled by predetermined change of composition.

Then came the Boulder dam project, the extensive cement studies by the Government, and the writing of the first specification to include composition.

If we now record that a modification of the Boulder dam specification is being written to meet less severe conditions, we shall have brought to date the contribution of the army of the center, the army of cement. The one group had taught us to design concrete; the cement group has balanced that contribution with advanced knowledge on composition.

Often-times the acceptance of new knowledge is distressingly slow; in fact, invariably so when large groups are concerned. In this case the group is small and the danger is that there may result a multiplicity of specifications, varying but little among themselves, not serving nor designed for distinctively different and unusual conditions. Such a development would interfere with the bulk production so essential for the economy and low cost upon which the great development of the concrete art has been based.

Therefore a new knowledge of cement is with us and that great achievement stands at the end of the path the army of cement has traveled. The knowledge is not complete—no knowledge ever can be in a progressing art—but a critical stride has been taken.

PROGRESS IN DESIGN

And now may we turn to the army of design, the army of the right?

Knowledge of reinforced concrete design was in its infancy in 1900. It was not until a few years later that Talbot's classical work was started at the University of Illinois. By 1910 only five major bulletins had been issued. But the work of Talbot and other investigators in the following five years, added greatly to the store of knowledge. Their work was indispensable and monumental. It dealt with the elements—columns, beams, footings—and we could go nowhere until we understood them.

Of quite another nature, however, was the assembly of these elements into a structure. Almost to this day, in the United States at least, the assembly has followed designs traditional in timber and steel. Each

element is considered independent of others—the post and lintel system merely applied to a new material. The great inherent advantage of concrete, that which marks it different from all other structural materials, which enables it to be adapted to any architectural form, which gives to it tremendous advantages in economy—that characteristic, continuity, has been neglected.

To this statement, one exception should be made; the flat slab, which uses this principle, is a contribution to the structural art made by American designers.

The reasons for the neglect of continuity are not hard to find. American engineers, generally, were not trained in the design of statically indeterminate structures, nor was their calculus adequate to the mathematics involved. Two contributions, the first in 1918 and the second in 1929, started the revolt against tradition by simplifying continuous design methods. The first was Maney's slope-deflection, and the second Cross' moment-distribution method. A third contribution was the improved training of engineering students in structural design.

The bridge field first felt the effect of the new methods. The rigid-frame bridge, bringing new economies in span lengths between 40 and 120 feet, began to be used in different parts of the country. Change was slow at first but the popularity increased until in 1934 there were some 150 built or put under way, as many as had been built in the previous 10 years.

Already it is clear that the principle, having been accepted for bridges, and much of the terror of design for continuity having been dispelled, a new era in reinforced concrete in this country has definitely opened. The age of concrete, in the structural field, is upon us.

CLOTHING THE FRAME

Note now, my friends, how the army of design is building to a magnificent climax in the structural art. First they had to discover the properties of the simple elements—beams, columns, footings—the alphabet of our structural language. They progressed to a combination of these in consonance with the inherent continuous character of the material. Had they stopped there, we would still have had only a bony framework for our structures.

But man, gifted with imagination, infused with a touch of the Divine in that he is made in God's image and likeness, yearns for the beautiful. Widely separated in this country were geniuses who were to take the final step, to see in our utilitarian, load-bearing material a substance of great potential beauty. The Franciscan Monastery and the

Church of the Sacred Heart, in Washington; the WCAU building, in Philadelphia; the Bahai Temple dome, in Chicago; the Department of Justice ceiling panels—will go down in the history of the art as the first final proofs that beauty has come to dwell with and in our material.

It is far in distance but not in inspiration from Washington to Los Angeles. While Earley was breaking paths in the East, a whole race of virile men was working creatively on the shores of the Pacific. Fertilized by their genius there seemed to rise from the very soil as an indigenous growth scores of buildings demonstrating a principle of transcendent importance in the long history of construction—that in concrete we have a material that at one and the same time is structural, enclosing and decorative. It requires no facing. It is its own facing material, cast integrally with and forming part of the supporting structure. And in this respect it is unique. No other structural material, with economy at least, combines those qualities.

Hardly less important is the development by these architects of the waste mold for casting ornament integrally with the supporting structure and their creation of textures of splendid variety. These are essential parts of their art and not separable therefrom, but they deserve separate and independent emphasis in order to give due appraisal to the extent of the contribution which these geniuses have made.

Such my friends, is the path followed by the army of design, no less brilliant in achievement than the accomplishments of their colleagues in concrete and cement. In many respects their accomplishments are far more dramatic, for concrete and cement are but the humble ingredients of the magnificent structures which the army of design erects. On all three paths there are triumphs of which any art may well be proud.

WHAT OTHERS HAVE DONE

We must, perforce of time limitation, come now to the end of our appraisal of the paths we have traveled. It is inadequate. It neglects much of importance. Particularly does it neglect an appraisal of the great educational contribution made by the engineering colleges, by your own Institute, by the American Society of Civil Engineers, the American Society for Testing Materials, the Joint Committees on Concrete and Reinforced Concrete, and the Portland Cement Association. That educational contribution has been an essential element in the advances we have sketched. It was necessary to the processes of improvement and creative thought, and to the acceptance by engineers,

architects and constructors which carried the improvements into the field and made them contribute to the benefit of the public.

But time forbids an adequate treatment. We must hasten io view our present position.

WHERE WE STAND TODAY

A proper plan for this paper, as suggested at its beginning, was to examine the path over which we have come, to assay our present position, and then look into the future. Concrete yesterday, today, tomorrow.

We have finished one part of our task, the survey of yesterday. What is our position today?

It seems to me that we have already answered that question, for we have defined the ends of our several paths. Possibly, though, a summary is called for, a brief statement of where we stand today. May I put it in terms of three coordinates:

1. A newly acquired and fairly complete knowledge of the design of concrete itself, together with machinery for accurately producing day after day the concrete we want;
2. A newly acquired and fairly complete knowledge of the constitution of cement;
3. A newly acquired technique of design for continuity and enough experience in the integration of structural, enclosing and decorative elements to give us a safe basis for sure and swift advance.

These are the co-ordinates of our position today, the base from which we set out for tomorrow.

ANSWER TO THE DIRECTORS

But before we face the future, let us pause and make answer to one of the questions put by the directors of this Institute. They asked whether the art was alert, sensitive to the need for change. The answer is yes, positively yes. But that answer, firm as it is, gives an inadequate idea of the virility, the flexibility, the alertness of our art. Let us be more specific and answer the directors in these words:

"Our art is alert. It is adapting itself quickly to new conditions. In fact, the curve of its progress is rapidly accelerating. The most recent ten years have been the most crowded of the thirty and four we have surveyed. Indeed, much of our knowledge is so new that we cannot truly appraise it. Only yesterday we learned to design concrete; the new knowledge of cement is of this morning; so, too, are the simplification of methods of continuous design, the coming of beauty, and the combination of structure, enclosure and decoration. Their real use is of tomorrow. Truly we are set for rapid progress."

Such is our answer to the query of our directors.

WHITHER ARE WE GOING?

And now may we turn to the future?

Prediction is always dangerous, but its hazards are lessened if we use a few principles. We may project the curve of the past as we have already done in hinting at sure and swift advance. Or we may apply the methods commonly used by every thoughtful man in developing his own character; he improves and strengthens his native good traits and corrects his defects. So, too, in concrete, can we not safely predict (1) that progress will be achieved along lines already clearly indicated and that we shall (2) strengthen its inherent good qualities and (3) overcome its defects?

Of progress achieved through overcoming defects, hardly any art can offer a more illuminating example than our own. Concrete is weak in tensile strength. That, in fact, is its chief defect and relegated it for centuries to use in a limited field. Then came Monier and the introduction of steel reinforcement. The chief defect was overcome. A revolution had been accomplished.

So, too, has progress been made by overcoming other defects. We have replaced ignorance of proportioning by knowledge. We have conquered low early strength. We have mastered high heat evolution in mass structures. We have replaced variability between batches by uniformity. So, too, shall other defects be cured and proportionate progress be registered.

We shall reduce volume change; we shall not be content with anything less than a shrink-proof, crackless concrete. Already there is such progress in this direction that this is no unguided boast. Brittleness may very well make way to a certain degree toughness, even as today we have breakproof laboratory glass.

Heat insulation—either alone or in combination with other materials (even as steel has been combined with concrete for tensile strength)—will be improved. Side by side with dense concrete which reflects sound, will be placed sound-absorbing concrete—a material of great technical and economic advantage in that it will combine sound-absorbing and structural properties. Already we are well along in this development. Its perfection is clearly in the offing.

Drab appearance, the work of untutored hands will give way to beauty. The technique has been well developed. Intensive educational work will spread its knowledge to constructors generally.

High form costs—a heavy present economic disadvantage—will yield to careful field research.

Lack of uniformity in the field will be remedied by persistent educational work which will result in putting in every sizable construction organization a thoroughly qualified technical man to design and control concrete mixtures. This day will be greatly hastened by the engineers' insistence upon accurate proportioning and rigid control.

In all of these directions, the overcoming of defects will give us a better material, will reduce its cost and will improve its service to the public.

IMPROVING ON GOOD QUALITIES

In the improvement of the inherent good qualities of our material, we may well place first the further increase of compressive strength. Rational proportioning sent us leaps forward in improving that native quality. The new advances doubtless will revolve for a while around vibration.

Does it require much imagination to see a new era of economy, wherein higher strength will be obtained and higher stresses be permitted, with resulting reductions in cross-sections and economies in construction. In combination with continuity of design, it will create a new era in building codes through which the economies effected by the new technology will be passed along to the public. We are now greatly over-designing our buildings. The factors of safety are needlessly high. With greater knowledge that era should be at an end.

Another of concrete's inherent advantages—its fire-resistive and fireproofing quality—has been so highly developed that progress will be largely in the way of widespread use, though doubtless improvements in the fire-resistive characteristics will also be made. The Chicago Stock Yards fire and the destruction with serious loss of life of the Kern Hotel, at Lansing, Mich., were 1934's way of telling us that in an automobile-airplane-radio-motion-picture age America is housing itself in structures of the horse-car vintage. We destroy right and left. Through insurance premiums and support of fire departments we tax non-combustible structures for the hazards imposed upon them by surrounding tinder. We can make our cities non-burnable, with great reduction in accident and loss of life and with large economies for the public at large. Can it be that the next decade will fail to bring the awakening? Ours is the fire-resistive and fireproofing material par-excellence, for it combines structural strength and fireproofing in the one material. In its use for that combined purpose we can predict marked development.

Many other inherent good qualities, impermeability, resistance to weather, to wear and to corrosive substances, all have shown improve-

ment and can be expected to register still further advances. Laboratories and constructors are at work in all these directions. The armies are still marching on.

THE FUTURE MARKET

Here may I pause a moment to answer those doubting Thomases who wonder whether there is an expanding future in store for the construction industry. They point out that our population will become stationary in not over 25 years, and they argue that a non-expanding population will not need an expanding physical plant. They conclude that the market for construction will gradually be restricted.

I do not share that fear for the immediate future. In my judgment, America must be rebuilt. True, in the heart of each one of our cities we have a reasonable number of fireproof, vermin-proof, termite-proof, low-maintenance cost structures. But sandwiched in between them are tinder of the worst kind, while our residential areas largely are of obsolete construction. America is too progressive to remain in that condition. Sanitary and social demands, if none others, will bring the cure. Already the matter of slum clearance has become a national issue.

Further, our wealth production is going to proceed apace, due to our increasing use of power and the development of new products and processes. The resulting wealth will find use, in part, in providing still more and greater structures for the convenience and comfort of our population; in new public buildings, educational, medical and recreation facilities; in better water supply and sewage disposal; in soil and water conservation; in improved highways and other transportation facilities; in more modern single and multiple housing. America will not be content with its physical plant until it comes up to the standards of the automobile age.

In my opinion, therefore, the employment of our industry in improving America's permanent plant will be intense as far ahead as any of us can see.

THAT WHICH IS HIDDEN

Nor can we gauge the future solely by what is plainly foreshadowed. There are in the womb of time more astonishing things than we can foresee. Great progress is by leaps, not alone by steady trend. So it was in physical science when the Curies discovered radium. So in steam power when the turbine was introduced.

So doubtless, will it be with concrete. There will be steady trends upward, but now and then will come great leaps forward, as radical as the insertion of a bar of steel into a mass of concrete. We cannot pre-

dict these leaps in kind or in time. But we can believe that science, which works wonders in other arts, will work wonders also in ours.

OUR TASK

It is in the service of the public through the use of this versatile material that we, of this Institute, are engaged. Shall we in our day make a worthy contribution? In the last three decades the difficulties in our art bred up armies of men competent to overcome them, headed by such leaders as Talbot, Turneaure, Hatt, Withey, Slater, Bates, Bogue, Earley, the Los Angeles architects, to name only a few.

They and their colleagues in Europe have in a single generation developed an engineering material from infancy to full stature, an extraordinary and probably an unique achievement in the history of engineering. That achievement properly ranks with the development of the automobile in rapidity and the extent of its contribution to the welfare of mankind. If it has failed to receive an equally great public acclaim it is only because its service is less spectacular and less obtrusive.

Happily many of the leaders responsible for this development are with us today. But when their individual work is done, the *great work* of which theirs is but a part must go on. The future calls loudly for worthy successors.

CONCRETE IN OUR LIVES

And what a material we have to work with! Because of its economy, strength, rigidity, resistance to abrasion, impermeability, fireproofness, skidproofness, sound-absorption, plasticity, adaptability to any structural or architectural form—all integrated into one—it has entered intimately into our community and individual lives. On the highway, it gives comfort and safety. In the school, the church, the theatre, it protects against fire. In sanitation, it enters into our water supply and sewage disposal structures, into the floors of meat packing and food processing plants. Even in recreation, it has its place—in swimming pools, tennis courts, playgrounds. If used in homes it insures fireproofness, rigidity, freedom from shrinkage, cracking and attack by vermin, and low maintenance cost. Truly it enters our very lives and is part thereof—serving humbly in foundations and roads, and then rising to glory in the dome of the Bahai Temple. Concrete the humble, can also be concrete the proud.

But, my friends, its true exaltation is still before us. I have already attempted to foreshadow it. It is in the architecture of tomorrow that we shall find its great glory. Already its progress is well advanced; for witness look to the Pacific Coast. Its march is irresistible, for here we

find in one material economy, plasticity, adaptability to any architectural form, integration of structural, enclosing and decorative elements; above all, beauty in whatever form the architect may desire —simple and impressive in well-balanced masses, rich with ornament cast integrally with the structure, textured to suit any whim of fancy or flight of the imagination, colored with the most delicate pastel or with the riot of the rainbow, the color and the texture themselves not added to but inhering in the structural and enclosing material.

Can you see it arise before you—the new architecture, the architecture of the plastic, the integral architecture, evolving out of this new medium of expression?

Finally, it is in the coming of beauty to concrete that I see the true guarantee of its future greatness. We are passing out of the pioneer stage. The organization of symphony orchestras in every part of the country signalizes the disappearance of the frontier. So, too, does the coming of beauty in the common things of life. Our automobiles are things of joy, true craftsmanship products. Our homes have repudiated the Victorian era. Beauty has entered even the kitchen. Into this *tempo* concrete has fitted itself; it is ready at hand to serve the craftsmen who will write into enduring structures the thought and the aspirations of a new age.

Why Continuous Frames?*

BY HARDY CROSS†
MEMBER AMERICAN CONCRETE INSTITUTE

INTRODUCTION

The problem of continuity was obvious from the earliest days of construction in reinforced concrete. In recent years it has increased in prominence and in importance. Constantly today some new form of continuous construction challenges the ingenuity of the analyst or offers new possibilities to the architect and structural designer. Many would like to know why the new types are being developed, what are their possibilities and what future developments may be expected in this field. It is difficult to answer all of these questions, but it may be profitable to look at certain phases of the subject.

Some structures are almost necessarily continuous and some structures become continuous by will of the designer. I wish to discuss chiefly the latter type but I shall refer somewhat to the former type also.

Let us recognize in the first place that continuity is not a new element in structural design nor is it peculiar to structures of reinforced concrete. The Baths of Caracalla in Rome are quite definitely continuous structures. Most construction in stone is continuous; Gothic cathedrals, the Renaissance stairways of France. You may ask how we can have flexural continuity in a structure built of stone without any tensile strength in the material. The answer is quite clearly that tensile resistance is developed in the material by adding weight; it is essentially a negative compression resistance, but acts quite as effectively as does reinforcement in a concrete structure. Buttresses supporting Gothic vaults, then, are quite definitely columns subjected to flexure.

Reinforced concrete as a material of construction started out with two obvious handicaps. In the first place, it is, like stone, much heavier in comparison with its strength than is timber or steel. Its weight eats up its strength in long-span construction. Moreover

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unless designed with skill, it looks heavy, has an awkward appearance, in long spans. If you doubt it, look at any of the through concrete girder bridges built twenty years ago on our highways.

CONTINUITY

Reinforced concrete is almost necessarily continuous. Continuity in construction is not by any means an unmixed blessing. We often go to considerable expense to avoid continuity in steel construction and we shall hear today from Mr. Moreell of expedients in the form of hinges devised to avoid continuity in structures of reinforced concrete. As many of you know, Prof. Wilson has recently completed at the University of Illinois extensive experiments to study the objectionable features of continuity between deck and superstructure in concrete arches. This continuity is by some designers carefully avoided by the use of joints. As we all know, continuity is a prolific source of cracking in many structures, in the walls of houses, in brick piers, in concrete floors, in the decks, gutters and handrails of bridges.

On the other hand, I need scarcely plead before this audience that concrete has many virtues. From the point of view of the architect, its leading virtue is plasticity; it can be molded. We can have any form we like, any shape, any curve. Note here that another older material of construction has now also gone plastic, for today we burn and weld steel into forms which could not have been considered some years ago.

Continuity may be a considerable nuisance to the designer but artfully handled it not only ceases to be a source of danger but in most cases points the way to overcoming the handicap of dead weight and massive appearance and lends itself to the full utilization of the plasticity of concrete by the development of more graceful forms of construction. Careful study, then, of this necessary nuisance will point the way to full realization of the full possibilities of reinforced concrete as an architectural material.

These possible advantages from continuity in structures of reinforced concrete are in many ways quite different from those in structures of steel. These two materials have aped each other too much. In structures of reinforced concrete, dead load commonly predominates as a source of stress much more than in structures of steel. We are consequently enabled to design concrete structures primarily for a single condition of loading and only incidentally for moving live loads. Too little attention seems to have been paid in structural literature to the great importance of the relative values of the source

of stress in comparing structural types or in comparing different structures of the same type but built of different materials.

STUDY OF A RIGID FRAME

I should like now to point out certain characteristics of a familiar structural type the design of which is commonly a routine matter and the analysis of which is often a professor's plaything. This is a simple, two-legged bent carrying vertical loads.

Assume that the girder has been designed as a beam simply supported. We wish to study the effect of flexure of the beam on the column stresses and of column flexure on the beam stresses, as the column depth varies. Assume for the present that the column width is constant and neglect for the present the effect of column reinforcement.

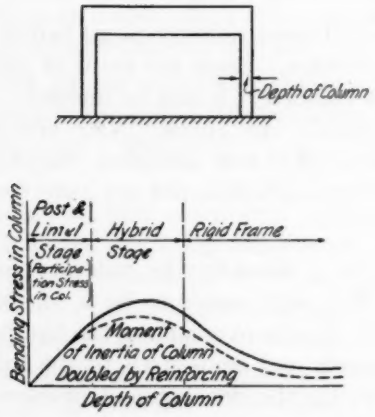

FIG. 1

We can distinguish clearly three quite different stages of action of this structure. Fig. 1 shows the curve of flexural stress in the column as the depth of column varies. You will note that if the column is very slender the flexural stress in it varies almost directly as the depth. The column here does not have to take a certain stress, it has to take a certain deformation, it has to bend through a certain angle. All we ask of it is that it shall bend through this angle without its main function as a compression member being impaired. It must, then, act as a hinge to the girder; of course the thinner this hinge, the

less dangerous it is. Strength in flexure has little or nothing to do with it; if we add reinforcement we increase its capacity as a column but we do not decrease appreciably its fibre strain as a hinge.

We are constantly calling upon concrete members—and steel and timber members too, for that matter—to act as hinges. They do it very satisfactorily. Usually we do not worry about it. But if the hinge is too thick—too stiff in the common loose sense of the word—we do need to worry. I shall return to this subject of hinges.

I like to call this stage of this simple problem the stage of Post and Lintel Construction. The columns here are subjected to participation stress—more accurately, to participation strain.

As the column depth increases a stage is reached where the flexural stress is uncertain. Increase in column depth may either increase the stress or decrease it. Approximately, however, at this stage the stress is independent of the depth. Clearly the column may no longer be considered as a hinge and it may be dangerous as a column; moreover, it is not helping the girder appreciably. I call this the Hybrid stage of action. If the stresses here are too high, it is difficult to relieve the situation without changing the general proportions of the structure; an increase of 100 per cent in moment of inertia secured by adding steel may reduce the stresses only 25 per cent.

Beyond this stage we have what may well be called a Rigid Frame. The moment carried by the column is now more and more nearly equal to the fixed-end moment in the girder. We can now directly proportion the member for a definite or nearly definite moment.

The structure is now a rigid frame, but it is a very poor rigid frame. We do not need all that concrete in the center of the span. It produces high dead load stresses to little purpose. So we core out the soffit of our girder as shown in Fig. 2, save material, reduce dead load and add to the gracefulness of the structure. We are now coming to the usual type of rigid-frame bridge and continuity is here pointing the way to economy of material and to improved appearance.

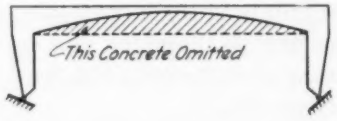

FIG. 2

How far can we go in reducing this center depth? Further than is customary in the design of rigid frame bridges if we wish to do so. As we continue to decrease the depth of this center portion of the girder, this portion becomes also a hinge.

If we go further and hinge the bases of the columns, the structure is a three-hinged arch and, except for the center hinge—quasi-hinge, if you prefer— the moments are very definite, they are determined by statics; we may design for them at once. So we have passed from one type of articulated or hinged structure, the forces in which are determined by statics—a beam simply supported by columns which act as hinges—to another type of hinged structure—a three-hinged arch. Just so architecture passed from post and lintel in the Parthenon to Gothic vaults with buttresses at Chartres.

QUASI-HINGES

Nowhere in this construction need we insert definite articulations, actual hinges. We may do so if we like but we do not need them if our quasi-hinges are properly designed. We may have if we prefer— and I often do very much prefer— essentially action of a hinged or discontinuous type in a structure which is everywhere continuous in its material.

It is not necessary to define with precision the degree of flexibility for which a thin section may be considered a quasi-hinge. The important concepts are (1) that by inserting in our structures sections that are relatively flexible we may make these structures act very nearly as statically determinate, articulated, structures, (2) that the angular deformation occurring at such quasi-hinges is, over a considerable range, nearly independent of the flexural resistance of the hinge. The second characteristic might lead to a specific definition, for example that doubling the section modulus of the quasi-hinge shall not reduce the angular deformation at the hinge by more than ten per cent, though in any given structure this would depend somewhat on the loading condition or source of deformation. The hinge action is a matter of relative flexibility of the hinge in a given structure and not one of absolute flexibility or definite thickness.

There is an important difference between subjecting a structural part to a load and subjecting it to a deformation. Many phenomena which tend to bring on failure in the former case tend to inhibit failure in the latter case. Specifically, cracking of a concrete beam tends to bring on failure due to applied moment but tends to delay failure due to an imposed angular rotation.

It may be well to caution against undiscriminating substitution of the viewpoint here presented for the more classical analytical view-

point. We shall still need the analyses but this viewpoint helps in interpreting them. We must also be cautious against a too ready acceptance of time flow in relieving the hinge because time flow, while it increases the flexibility of the hinge, also increases the angular deformation to which it is subjected as an effect of loads. Departure from Hooke's Law is, however, a very dependable source of relief in any case and time flow is a dependable source of relief for deformations not due to loads.

But we need to know more about these quasi-hinges. Laboratory experimentation in the past has concentrated its attention on the strength of concrete members. We need from the laboratories more information on the permissible deformations of members of reinforced concrete. Such studies have recently been published in steel; Professor Parcel has shown us that the hinge action—secondary stresses—of the members of a steel truss need not in many cases be a matter of serious concern. We know that often concrete columns are subject to considerable deformation without serious damage. But we need more definite information as to desirable type of reinforcement, shape of hinge, manner of disintegration for large deformations, strength against direct and shearing forces after some disintegration has occured as a result of large angular deformations.

The quasi-hinge at the center of the rigid frame is a very different hinge from that at the ends of the simple girder. For the columns supporting the simple girder we may see that for a given angle change in a given length, the strain in the outer fiber is proportional to the depth of the member. But the hinge at the center of the rigid frame is a tapered hinge; the greater its center depth the greater the length brought into bending. In this hinge, then, we find that the strain in the outer fiber for a given angular rotation is more nearly independent of the depth at the center.

DEFORMATION STRAINS

All of these frames—post and lintel type, hybrid type, rectangular rigid frame type, rigid frame with curved soffit—are, of course, subject to what I like to call deformation strains—strains produced by shrinkage, by abutment movements, by changes of temperature. These strains may be insignificant or they may be serious. In many cases they are a source of much concern to engineers. If the structure is of a statically determinate type—post and lintel or three-hinged arch—we need concern ourselves only with the action of the quasi-hinges. These quasi-hinges are subject to a fixed deformation and the unit fiber strain will depend on the amount of angular rotation and on the

shape and depth of the hinge. But here again as we approach the discontinuous hinged type, the action of the main structural frame is pretty definitely determined.

RIGID-FRAME BUILDINGS

We may consider another very similar type of rigid frame which has large possibilities, practical and aesthetic. This is the rigid frame building, Fig. 3. This structure is designed essentially for gravity loads; in general, wind loads do not affect the design and there are either no moving loads or they are small.

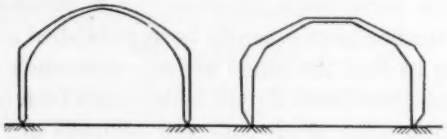

FIG. 3

The question at once forms, How does such a structure act? A more significant question is, How do you want it to act? It may be anything between a post and curved lintel—simple beam—construction and a three-hinged arch as the designer chooses. We may within these limits draw any curve of moments we wish consistent with the loads and then so proportion the structure that this will be the true curve of moments. We may do this by varying the moment of inertia along the frame.

What do we want here? Since the chief function of the material in the frame is to carry dead load, we may expect maximum economy by reducing and re-arranging this dead load. It is most economically carried to the footings by direct compression in the columns, not by bending in the roof girder. Hence the three-hinged arch construction is indicated as the desirable form, the shape approaching as nearly as architectural requirements permit to a pressure line for the dead load of the structure itself.

ECONOMY FROM REDISTRIBUTING DEAD LOAD

The economy to be realized by re-arrangement of dead load may be suggested by study of a beam fixed at ends. It is true that there is no such thing as a beam absolutely fixed at ends but we do approach such a condition in the case of the ordinary rigid-frame bridge. Suppose that the beam is to be designed for fixed maximum fiber stress and is to carry only its own dead weight.

Two designs are compared in Fig. 4. In one the beam is of constant section throughout. In the other, the soffit tapers as a parabola to a very small depth at the center. Neglect any slight arching effects. The amount of material required in the second case is only 1/12 of that in the first case, and the amount of reinforcement is of the order of ¼ as much. The conclusion here is not to be taken too seriously for there is no live load involved but the values at least challenge attention.

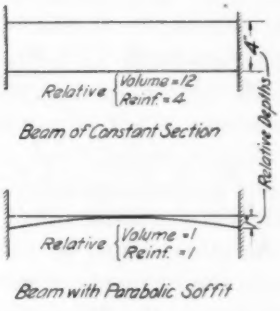
4
Relative { Volume = 12 / Reinf. = 4
Beam of Constant Section
Relative Depths
Relative { Volume = 1 / Reinf. = 1
Beam with Parabolic Soffit

FIG. 4

ORDINARY BUILDING FRAMES

Ordinary building frames are necessarily continuous; continuity is essential to their stability under the action of horizontal forces. We have found it most convenient here to use members of constant section. Much of the dead load is in the slabs and haunching to redistribute the dead load presents less attraction than in the case referred to above. An important problem now before the profession is whether or not we shall depend upon the column flexure to relieve the moment in the girders as we did in the rigid frame bridge just discussed. You will remember that the First Joint Committee did not permit this but that our present code permits it in a somewhat arbitrary fashion in certain cases. Some now wish to do it in all cases and in a systematic way rather than in an arbitrary way. I may say here that I strongly favor this movement not because I see that it will lead immediately to any great improvement in our buildings so long as we stick to present forms, with beams and columns of constant section, but rather because I have a profound faith in the ingenuity of structural designers when they are permitted to exercise their ingenuity.

Some uncertainties are inevitably involved in this procedure. It is of interest to go back to our simple bent. The flexural stresses in a column may be written in a very familiar form as

$$M\frac{c}{\Sigma aI}, \text{ where}$$

M is the fixed-end moment at the joint.

c has the usual meaning in flexure formulas, distance from neutral axis to outer fiber.

ΣaI is the sum of the adjusted moment of inertia of all members meeting at the joint, the factor a depending on the conditions of restraint, upon the relative length of the members, and upon the condition of loading.

The value of f thus computed may be further modified to allow for reduction of moment to the bottom of the girder and for the cracking of the section. This, however, is not important for our present purpose. The important relation brought out is that the moments of inertia of both column and girder are involved and that extreme precision in the computation of one is futile unless we can secure precision in the computation of the other. But, as we all know, there is considerable uncertainty as to the moments of inertia of the girders.

As nearly as I can determine, the flexural stresses indicated in the wall columns of ordinary buildings may be of the order of 300 to 400 p.s.i. In general the frames are in the hybrid stage of construction indicated above for the rigid frame and neither increase in column diameter nor added reinforcement affects very much the flexural stresses. Reduction must be made then for these stresses in determining the stresses available to carry direct load.

In ordinary building frames continuity of girders with adjacent girders is an obvious factor for which we have always made some provision in our designs. Continuity of girders with columns is something of a nuisance to the designer but he can get from it some benefit if he clearly recognizes it and allows for it in his design. Haunching promises some economy in these structures but has often serious architectural disadvantages.

In other types of concrete construction, building frames and bridges of long span, continuity points the way to a discontinuous structure with redistribution of dead load and consequently to economy and added gracefulness. By study of the possible discontinuous types the concrete designer may realize the full possibilities of the remarkably plastic material with which he has to deal.

CONCLUSION

Reinforced concrete has entered upon a new phase. At first, so many detailed problems of research crowded on our attention that it

was difficult for us to realize the full possibilities of this type of construction. Many and probably most of the more pressing problems of research have been solved, at least tentatively. In the past, the designer has often found himself restricted to standard structural types, fearing to depart from them because of the maze of uncertainties constantly raised by the questioning attitude of the research man.

But concrete today has passed out of leading strings. It is a plastic material, a material for an artist. We can build with it as we wish, of new forms, with new proportions and to new spans. We must encourage the designer to do so. We must encourage him by liberalizing our codes and specifications and by developing a different point of view in our texts.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32) Discussion should be available to the Secretary by July 1, 1935*

Articulations for Concrete Structures—the Mesnager Hinge*

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The utility of hinges or "articulations" for concrete structures has long been recognized; the difficulty has been to devise a hinge which could be installed at reasonable cost, which would not lose efficiency under service conditions, and whose behavior under working loads could be predicted within reasonable limits.

Hinges in concrete structures, such as arches, have a special value. By fixing the line of pressure at definite points and by permitting unrestrained adjustment with volume changes, the hinges counter-act uncertainties arising from the lack of complete knowledge of the behavior of concrete under variable loads, temperatures, and humidities. Where slight foundation movements may occur the stresses produced by these movements, as well as the so-called "parasitical stresses" (caused by shrinkage, plastic flow, rib-shortening, and temperature and humidity variations) can be reduced greatly, or, in the case of the three-hinged arch, eliminated almost entirely. There may result an appreciable reduction in the sizes of the sections, with consequent reductions in the cost of the superstructure and foundations.[1]

Early attempts to articulate concrete arches followed the precedents of steel construction. Steel seats, attached rigidly to the concrete on each side of the hinge, were designed to rotate on a solid steel pin. This type of hinge is costly and requires careful construction to obtain accurate alignment of the hinge over the entire width of the arch rib. A slight initial mis-alignment or subsequent corrosion of the hinge contact surfaces might effect materially the efficiency of the hinge and the stability of the structure.

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*This subject—not precisely this paper—was presented by the author at the 31st Annual Convention, American Concrete Institute, New York, Feb. 19-21, 1935.

The opinions or assertions contained herein are private ones of the writer and are not to be construed as official or reflecting the views of the Navy Department or the naval service at large.

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[1]For a statement of the advantages of hinged masonry arches, see *Proceedings A. S. C. E.*, June 1898 "Three-Hinged Masonry Arches; Long Spans Especially Considered," by David A. Molitor.

Another type of articulation has cylindrical contact surfaces of cut stone or of heavily reinforced concrete, the convex surface of smaller radius rolling in a concave surface. These hinges are difficult to construct and are of doubtful permanence as any foreign matter which lodges in the hinge could reduce materially its effectiveness.

An articulation used frequently in Germany was made by inserting sheet lead between the masonry contact surfaces at the hinge. The objections to this type are doubtful durability under severe exposure and service conditions, uncertainty as to the exact position of the line of pressure, and weakness in resistance to large shearing forces.

Considère used closely-spaced straight steel bars embedded in concrete with or without spiral reinforcement as a temporary hinge to take care of initial shrinkage, initial displacement of supports, and dead load rib-shortening. As this type of hinge does not resist effectively large shearing forces resulting from unsymmetrical loadings, the opening at the hinge is closed with reinforcing steel and concrete before the structure is subjected to live loads. The structure thereafter acts as a rigid arch. This type has been used in the United States as a temporary hinge in several instances.

Fig. 1 shows line diagrams of these types of hinges.

The Mesnager hinge or "semi-articulation" was developed by the late Augustin Mesnager, eminent French engineer and pioneer in concrete research. Mesnager reasoned that since the angles of rotation at the hinges in a concrete structure are very small, by crossing steel bars at an opening in the concrete he would obtain, at reasonable cost, an articulation which would offer very small resistance to rotation, would resist effectively either axial thrust or shearing forces, and would have permanent flexibility. A line diagram of the Mesnager hinge is shown in Fig. 2. Mesnager stated that since the lengths BD and B^1D^1 remain unchanged, the rotation occurs about the intersection C, which is the instantaneous center of rotation of B B^1 with reference to D D^1.

The dash line in Fig. 2 indicates the concrete cover placed over the bars at the opening to prevent corrosion. The effect of this cover on the strength and flexibility of the hinge will be discussed later.

To verify his theories, Mesnager made a series of tests on full-size hinges.[2] The type of specimen and arrangements in the testing machine are shown in Fig. 3. Two specimens were made, each 16 in. square and reinforced with 16 0.8-in. diameter bars. The first specimen was given a hinge rotation of 0.02 radians by inserting a steel

[2]See "Experiences sur une semi-articulation pour voutes en Béton Armé," by M. Mesnager, *Annales des Ponts et Chaussees*, 1907, II, p. 180.

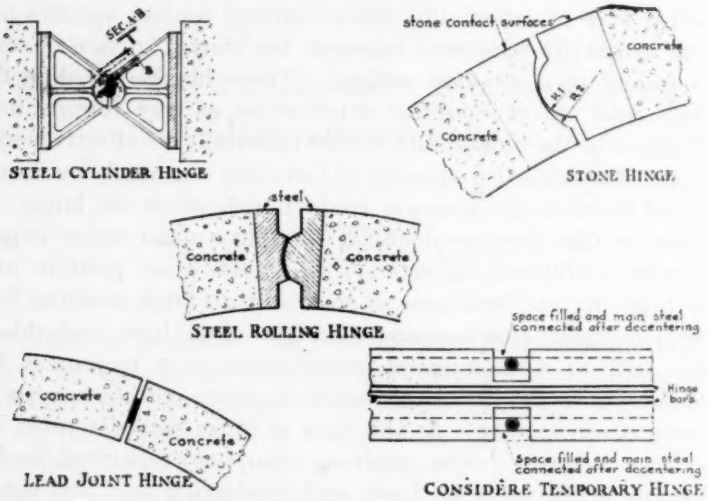


FIG. 1. TYPES OF HINGES.

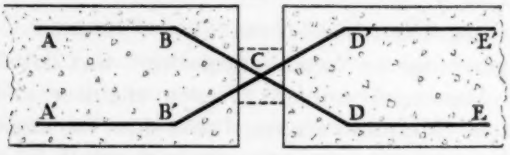
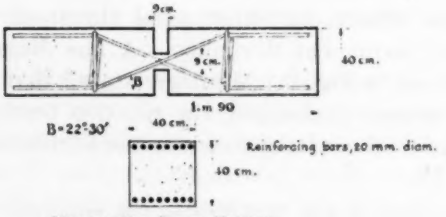


FIG. 2. SEMI-ARTICULATION OF MESNAGER



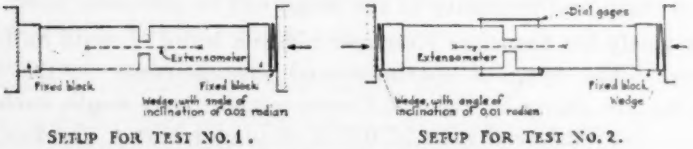


FIG. 3 MESNAGER TEST HINGE

FIG. 1, 2, 3

wedge between one end of the specimen and the platen of the testing machine. While the specimen was in this rotated condition, the load was applied until failure occurred. Failure was caused by cracking of the concrete blocks along the line of the steel, followed by slipping of the bars, the section at the hinge opening being undamaged. The breaking load was 323,000 lb., corresponding to a unit direct stress of approximately 44,000 p.s.i. in the hinge bars or slightly more than the yield point of the steel. In this computation, no account is taken of the strength of the concrete in the hinge opening or of the bending stress in the bars caused by rotation and thrust.

The second specimen was subjected to the same hinge rotation by inserting steel wedges of 0.01 radians at each end of the specimen. A load of 154,000 lb. (about half of the breaking load under the first test) was applied, released, re-applied and released. The steel wedges were then reversed to cause rotation in the opposite direction and the same loading was again applied twice. No evidence of failure resulted from these loadings.

The concrete cover over the bars was then removed and the same series of loadings was repeated without evidence of failure. The bars had a direct stress of about 21,000 p.s.i. under these loadings. The specimen was then loaded to destruction while in a rotated condition (with a rotation of 0.02 radians) and failed under a load of 238,000 lb., corresponding to a direct stress of 32,000 p.s.i. (75 per cent of the yield point) in the steel at the hinge opening. However, primary failure occurred in the concrete blocks and resulted from insufficient lateral reinforcement near the hinge opening.

Mesnager also made tests to determine the resistance to buckling of bare hinge bars when in a rotated condition.[3] Previously, he had made an extended study of the utility of similar hinges made of flexible steel plates which were substituted for the pin connections of steel trusses. This study included stress measurements on a steel railroad truss built with these joints.[4]

A complete discussion of Mesnager's work would be too lengthy for this paper. As a result of his studies and experiments, he advanced the following conclusions and recommendations:

(a) With a rotation of 0.02 radians or less, bare hinge bars can be counted upon to develop their tensile yield point in resisting the direct stress caused by thrust and shear. On this basis, for steel with a yield point of 41,000 p.s.i. and with a factor of safety of about 3.6, he recommends a working stress in direct compression of 11,300 p.s.i. By inference, he states that if the rotation does not exceed 0.02 radians, and with an l/r ratio between 20 and 40 for the bars across the hinge, no

[3]See "Cours de Béton Armé," by M. Mesnager, 1923, p. 184. Dunod, Paris.
[4]See *Annales des Ponts et Chaussees*, 1896, II, p. 750; id 1898, II, p. 300; 1899.

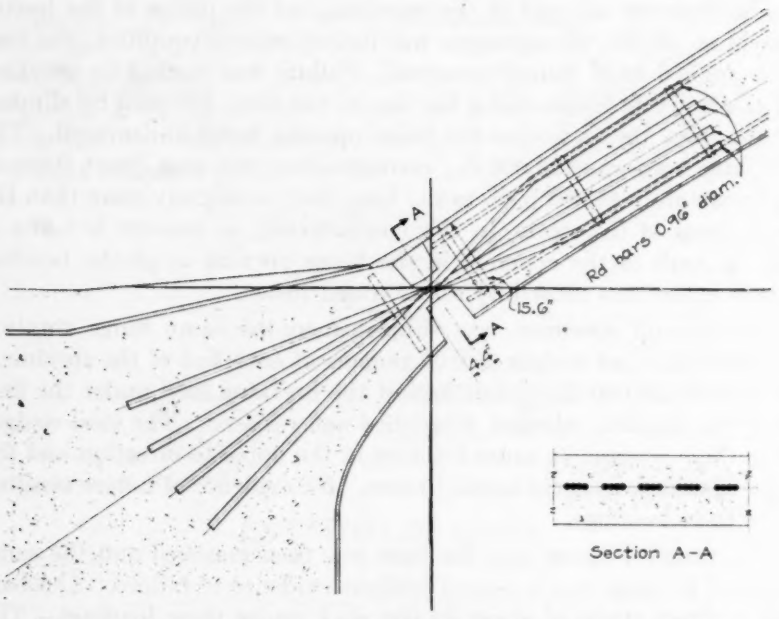


FIG. 4—DETAIL OF ABUTMENT HINGE FOR CANAL ST. MARTIN COVERING

account need be taken of the bending stress in the bars caused by rotation, thrust, and shear. He states that a rotation of more than 0.02 radians will be encountered rarely in practice.

(b) He recommends for plain bars an embedment of 45 diameters on each side of the hinge.

(c) The angle of inclination between the bars should be such that the line of thrust at the hinge will be well within this angle for all conditions of loading.

(d) The l/r ratio of the bars across the hinge opening should not exceed 40.

The l/r ratio in his tests was 21. With this slenderness ratio and under the working stress recommended, he obtained an eccentricity of the thrust at the hinge of about 1½ in. for a rotation angle of 0.0065 radians when the hinge bars were covered with concrete. When the hinge bars were bare, the eccentricity was reduced to about 0.15 in.

Mesnager recommends that the hinge bars be designed as though they had no concrete cover, the cover being considered only as a protection against corrosion, although it adds materially to the strength of the hinge. When it is desired to reduce the thrust eccentricity to a minimum, the concrete cover can be replaced by paint or other protective coating. In any event, to avoid large eccentricities, the total height of concrete in the hinge opening should not exceed the width of the opening.

In practical applications of the Mesnager hinge the bars are frequently placed in more than two planes, as indicated by Fig. 4, which is a detail of the abutment hinge used in the roof covering over the Canal St. Martin, Paris. He states that such a distribution of the bars results in a more effective transfer of the stress from the hinge bars to the concrete of the arch rib and makes possible a greater number of bars in a limited space.

Some examples of practical applications of the Mesnager hinge with references are as follows:

(a) Roof cover over the Canal St. Martin, Paris—a three-hinged arch with span of 90.5 ft., supporting an earth fill and a highway. Rf.—*Le Genie Civil*, July 30 and Aug. 6, 1910.

(b) Hippodrome at Lille, France. Cantilever canopy for grand stand with span of 83 ft. Three-hinged arch structure with 65-ft., span for exhibition hall. "Beton Arme" by Espittallier, 1932, p. 321. Published by Ecole Speciale des Travaux Publics, Paris.

(c) Municipal swimming pool at Paris. Three-hinged roof arch with span of 52 ft. *Le Genie Civil*, Apr. 20, 1929.

(d) Highway bridge at Termast, Morocco. Three-hinged arch span of 118 ft. Cantilevered abutments. *Le Genie Civil*, Mar. 2, 1929.

(e) Exposition building at Lausanne, Switzerland. Two-hinged arch with span of 110 ft. *La Technique Moderne*, T. XIII, No. 1.

(f) Bridge at Montauban, France. Two three-hinged arches with 175 and 185 ft. spans carrying steam tramways and a highway. *Le Genie Civil*, June 29, 1916.

(g) Dirigible hangar at Montebourg, France. Three-hinged roof arch with span of 81 ft. *Le Genie Civil*, Sept. 6, 1919.

(h) Highway bridge at Amelie-les-Bains, France. Three-hinged arch with span of 140 ft. *Le Genie Civil*, Aug. 27, 1910.

(i) Bridge over the River Noce, Province of Potenza, Italy. Three-hinged arch with span of 98 ft. *Annales des Ponts et Chaussees*, 1917—III.

In all of these structures Mesnager hinges were used as permanent articulations. This hinge has also been used as a temporary articulation in the same manner as the Considère hinge.

Mesnager has stated that, under competitive bidding, various arches constructed in France using this type of hinge have shown a saving of 20 to 25 per cent over rigid arch designs.

TESTS AT THE NATIONAL BUREAU OF STANDARDS

As indicated previously, in Mesnager's tests failure occurred either by slipping of the bars or because of insufficient lateral reinforcement in the concrete blocks. As a result, no test data were available on the strength of the bare bars across the hinge opening. In addition, no tests were made to determine the effectiveness of the inclined bars across the hinge opening to take care of shearing stresses, this feature not having been covered in Mesnager's tests.

In order to cover these points and to confirm Mesnager's conclusions, loading tests of Mesnager hinges were made recently at the Bureau of Standards. These tests were made for the Navy Department in connection with the design of a three-hinged arch roof for a ship-model testing plant.

A complete description of the tests, with an analysis of the stress conditions in the hinge bars, is given in the paper by D. E. Parsons and A. H. Stang of the Bureau of Standards.[5] In the following dis-

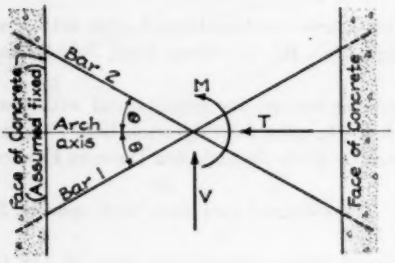


FIG. 5—LINE DIAGRAM MESNAGER HINGE

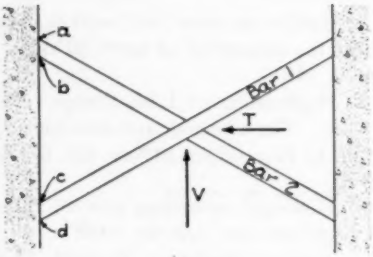


FIG. 6—POINTS OF MAXIMUM STRESS

cussion, it is assumed that the reader is familiar with the data presented in that paper. Where not here defined, all symbols are as defined in the Parsons-Stang paper.

Fig. 5, reproduced from the Parsons-Stang paper, shows a line diagram of the Mesnager hinge. In the general case, the hinge bars are subjected to a direct axial thrust T, a shearing force V, and the moment required to cause rotation at the hinge. The forces T and V cause direct stresses and bending stresses in the bars. To these stresses must be added the bending stress resulting from rotation. Secondary effects result from eccentricity of the thrust caused by bending of the hinge bars.

By Formulas 12 and 13 of the Parsons-Stang paper, there can be determined the maximum unit stress in the hinge bars under any combination of T, V and ϕ, the angle of rotation at the hinge.

Referring to Fig. 6, the maximum stress may occur at point a, b, c, or d of the hinge bars, depending upon the relative values of the V/T and l/r ratios and on the angle Θ. Assuming that $\phi = 0$, (no rotation at the hinge), the total combined stresses at these points will be—

[5] "Tests of Mesnager Hinges," JOURNAL AMER. CONCRETE INST., January-February, 1935; *Proceedings* Vol. 31, p. 304.

$$\text{at point (a), } \sigma_m = \frac{T}{2A}\left\{\frac{K}{C} - \frac{l}{rs}Q + \frac{V}{T}\left[\frac{L}{S} + \frac{l}{rc}R\right]\right\}$$

$$\text{at point (b), } \sigma_m = \frac{T}{2A}\left\{\frac{K}{C} + \frac{l}{rs}Q + \frac{V}{T}\left[\frac{L}{S} - \frac{l}{rc}R\right]\right\}$$

$$\text{at point (c), } \sigma_m = \frac{T}{2A}\left\{\frac{K}{C} + \frac{l}{rs}Q + \frac{V}{T}\left[-\frac{L}{S} + \frac{l}{rc}R\right]\right\}$$

$$\text{at point (d), } \sigma_m = \frac{T}{2A}\left\{\frac{K}{C} - \frac{l}{rs}Q + \frac{V}{T}\left[-\frac{L}{S} - \frac{l}{rc}R\right]\right\}$$

The first and third terms in each of the foregoing formulas represent the direct stresses caused by T and V, respectively; the second and fourth terms, the bending stresses caused by T and V, respectively. It is not necessary in any case to compute all four stress values as the charts, Fig. 7 to 12, can be used to determine the maximum stress for any particular values of l/r, V/T and Θ. To the stress determined by means of the charts there must be added *numerically* the fiber stress caused by the rotational moment, which is $2\frac{Er\phi}{l}$.

The stresses thus determined include direct stress plus superimposed bending stresses. However, super-position of the bending stress caused by rotation, thrust and shear on the direct stress, as commonly practiced for members subjected to bending and direct stress, will not give an indication of the ultimate strength of the hinge bars because, for large rotations, the most highly-stressed fibers of the bar are outside of the elastic range. For example; Hinge No. 5, of the Bureau of Standards tests, failed at 60,000 lb. under a rotation of 0.026 radians. The l/r ratio was 39.6. The direct compressive stress in the bars at failure was 45,500 p.s.i. and the maximum bending compressive stress under elastic deformation would have been 46,500 p.s.i. The sum of the direct stress and bending stress would, therefore, have been 92,000 p.s.i. which is greatly in excess of the yield point strength of the bar (47,000 p.s.i.) It is evident that when the most highly stressed fibers reached the yield point, a redistribution of stress occurred as the result of plastic flow in the steel.

The conclusions given above relative to the strength of the hinge bars under combined stresses are corroborated by the results of a recent study of the effect of secondary stresses upon ultimate strength by Professor Parcel of the University of Minnesota. (See *Proceedings A. S. C. E.*, Nov. 1934.)

Therefore, in the light of the test results, it is believed that even with a rotation as large as 0.026 radians, the bare bars across the

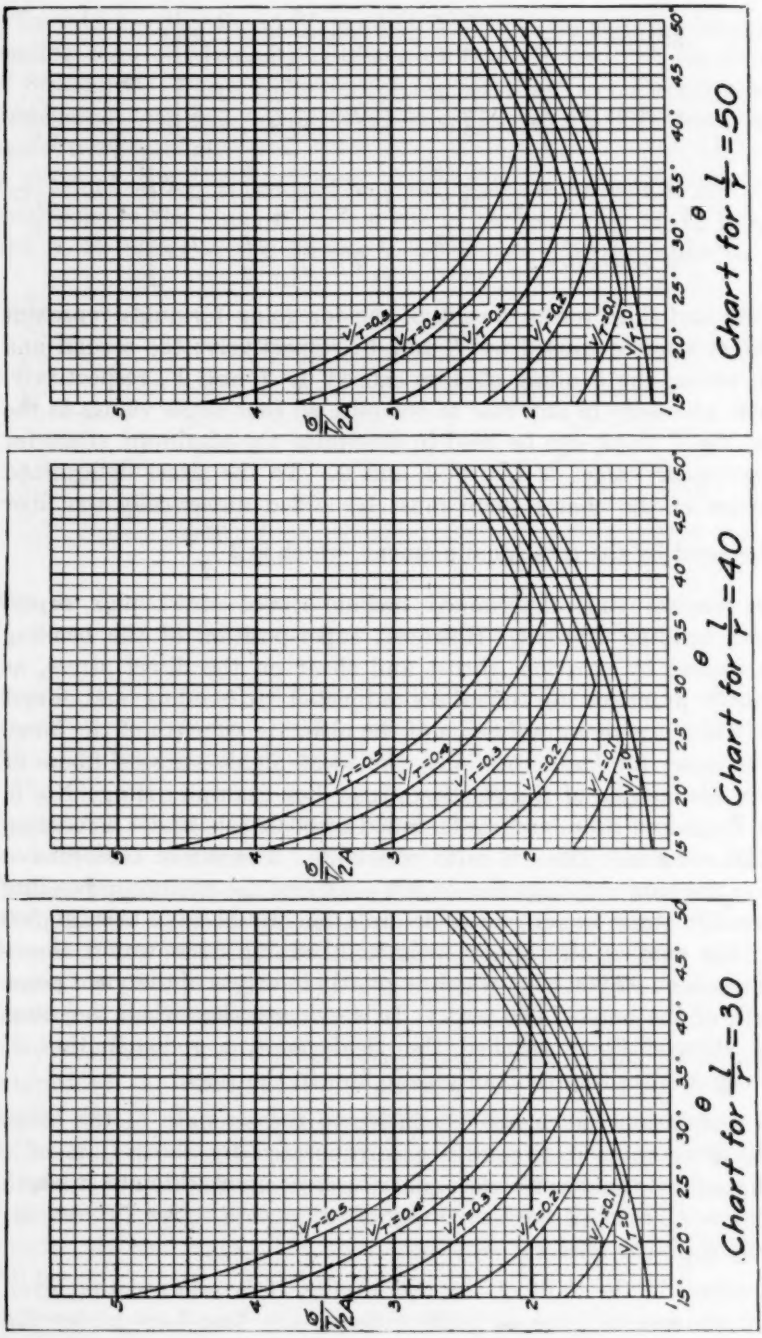


Chart for $\frac{1}{r}=50$
Chart for $\frac{1}{r}=40$
Chart for $\frac{1}{r}=30$
θ
$\frac{S}{T^2A}$
$V_T=0.5$
$V_T=0.4$
$V_T=0.3$
$V_T=0.2$
$V_T=0.1$
$V_T=0$

FIG. 7, 8, 9

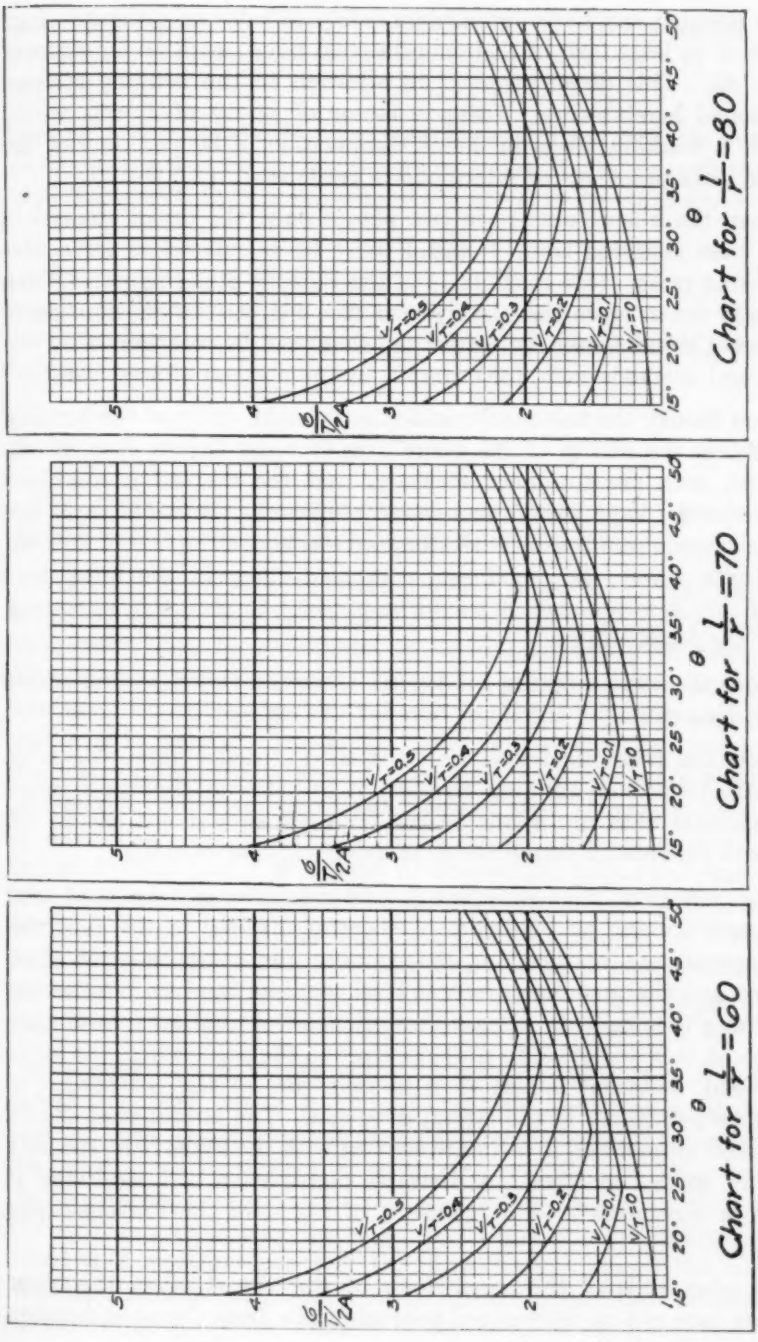


Chart for $\frac{l}{r}=80$
Chart for $\frac{l}{r}=70$
Chart for $\frac{l}{r}=60$
θ
$\frac{\theta}{\pi/2A}$
$v/T=0.5$
$v/T=0.4$
$v/T=0.3$
$v/T=0.2$
$v/T=0.1$
$v/T=0$
15° 20° 25° 30° 35° 40° 45° 50°
1 2 3 4 5

FIG. 10, 11, 12

hinge opening can be counted upon safely to take an ultimate *direct* thrust of at least 0.90 of their tensile yield point, with an l/r ratio of 30 to 40. This direct stress is in addition to the bending stresses mentioned hereinbefore. Using a factor of safety of 3, this would permit a working stress in direct compression of 30 per cent of the tensile yield point or, for intermediate grade steel, 12,000 p.s.i.

When the hinge bars are in two planes only, the direct stresses in these bars resulting from T and V may be calculated as for a pin-connected truss. (See formula 14 of the Parsons-Stang paper.) When the bars are in more than two planes (see Fig. 4) the distribution of the direct stress caused by thrust and shear can be determined by one of several methods such, for example, as the method of least work.

Even though the test results indicate the small effect of the bending stresses on the ability of the hinge bars to resist thrust, it is recommended, as a precautionary measure, that the maximum combined unit stress be kept within the elastic range. In the usual case, this can be accomplished easily with standard steels. In the case of unusual structures where the maximum combined stress in the hinge bars varies over a wide range, it would be prudent to keep this stress well below the fatigue limit.

Under the usual working conditions, the secondary buckling effects caused by bending of the hinge bars may be neglected.

While the theoretical analysis indicates the desirability of a large l/r ratio to reduce bending stresses, it is believed that until more experimental data are available the l/r ratio should not exceed 40. This will provide an ample factor of safety against buckling.

It is important that sufficient lateral reinforcement be used near the hinge opening to take care of shearing stresses in the concrete. An approximate method for determining the magnitude of these stresses and the amount of lateral steel required is given by Parsons and Stang (See p. 318). Regardless of the steel used for lateral reinforcement, it is advisable to provide, close to the point where the hinge bars bend, sufficient lateral steel to take care of the component of the hinge bar thrust in the lateral direction. The lateral steel should be hooked around and in intimate contact with the hinge bars, securely wired or welded in place. A desirable distribution of lateral steel is shown in specimens 3 to 7, inclusive, of Fig. 1 of the Parsons-Stang paper.

The magnitude of the eccentricity of the thrust when the hinge bars are bare can be determined approximately from the consideration

that the moment M necessary to achieve a rotation ϕ at the hinge results from a thrust with eccentricity, e, such that—

$$M = Pe,$$

where P is the magnitude of the total thrust. The rotational moment, M, necessary to bend the hinge bars in an arc of a circle such that the angle subtended by this arc will be ϕ is given by the formula—

$$M = \phi\Sigma\frac{EI}{l}$$

for all the bars across the hinge opening, where E, I and l, have the usual significance. Therefore, the eccentricity is obtained from the formula—

$$e = \frac{M}{P} = \frac{\phi\Sigma\dfrac{EI}{l}}{P}$$

The theoretical eccentricities computed by this formula were checked within satisfactory limits by the test results.

When the hinge bars are covered with concrete or mortar, the total height of which does not exceed the width of hinge opening, the eccentricity may be considered to be approximately 10 times that when the hinge bars are bare. The stiffness ratios determined by the Bureau of Standards varied from 7.5 to 11.5 while Mesnager found this ratio to be 9.5. The hinge bars might well be left bare where small eccentricities are desired. In such case, the bars should be protected by paint or corrosion-resistant steel should be used. As an alternative, the hinge bars might be left bare for a period long enough to take care of dead load rib-shortening, initial shrinkage, and initial abutment displacements. Where cover is used, the height at midpoint of the hinge opening should, if practicable, be limited to 0.75 of the width of hinge opening to avoid large eccentricities. In the usual case, the effect of the eccentricities at the hinge on the stresses in other parts of the structure will be negligible.

It should be noted that while Mesnager recommends the use of bare hinge bars where small eccentricities are desired, structures actually built have had cover over the hinge bars in all cases. Where the larger eccentricities will have no material effect on other parts of the structure, it is desirable to take advantage of the added strength and rigidity provided by the cover over the bars. Parsons and Stang found that mortar cover over the hinge bars increased the ultimate strength from 2 to 3.2 times. However, it is believed that no allowance should be made for the strength of this cover in designing the hinge.

This is a precaution which is justified by the questionable duration of the strengthening effect of the concrete cover under reversals of stress, shrinkage effects, and plastic flow.

For any particular value of the slenderness ratio (l/r) and ratio of shear to thrust (V/T) there is one value of Θ, the angle between the bars and the axis, for which the critical stress in the hinge bars is a minimum. Fig. 7 to 12, inclusive, can be used to determine the maximum unit stress in the hinge bar resulting from V and T, for values of l/r from 30 to 80, and of Θ from 15° to 50°. These charts can be used also to determine the optimum value of Θ for any given set of conditions. As stated hereinbefore, to the maximum stress thus determined there must be added (numerically) the rotational moment stress.

SUMMARY OF DESIGN PROCEDURE

The magnitude and direction of the thrust, as well as the rotation angle at the hinge may be determined on the assumption of a frictionless hinge. Having determined V, T and ϕ a trial value is selected for the l/r ratio and the optimum value of Θ is determined from Fig. 7 to 12. In selecting the value for Θ care should be taken that the extreme positions of the thrust line at the hinge lie within the extreme hinge bars. The direct stress in the bars can then be determined from formula 14 (Parsons-Stang) if the hinge bars are in two planes, or if in more than two planes, by the method of least work. The direct unit stress should not exceed 30 per cent of the yield point.

The maximum unit stress in the hinge bars can then be determined using the appropriate chart, Fig. 7 to 12. When the hinge bars are in more than two planes, the bars should be placed symmetrically about the axis, as shown in Fig. 4, and symmetrical pairs should be considered in using the charts. The ordinates in these charts are coefficients by which the factor $T/2A$ should be multiplied to obtain the maximum unit stress σ_m. For ordinary structures having a relatively small range of stress with infrequent fluctuations this stress (σ_m) should not exceed the yield point. For structures where the stress fluctuations are large and frequent it is desirable to keep σ_m well within the fatigue limit.

Lateral steel to take care of shearing stresses and the lateral component of the hinge bar thrust should be provided as described hereinbefore.

The bars should have sufficient embedment on each side of the hinge opening to develop their strength in bond.

The eccentricity of the thrust at the hinge and its effect on the stresses in other parts of the structure should be determined.

Hinge bars having a high yield point and great ductility would be an ideal combination, but it is an impossible one with ordinary carbon steels. The use of intermediate grade reinforcing steel is recommended except where the importance of the project warrants the use of chrome-nickel steel. Certain grades of the latter have a high yield point combined with great ductility and have the added desirable property of retaining their ductility at very low temperatures.

While experimental data on Mesnager hinges are not sufficiently extensive to warrant the assertion that our analytical studies have been corroborated for all possible combinations of the variables involved, it must be recognized that this type of articulation has been used successfully for important structures of considerable magnitude and that devices designed for analogous purposes have in some instances been used with a less extensive background of experimental research. The writer is of the opinion that more complete studies of this articulation in laboratory and field should be made and are warranted in view of the successes already attained in its use.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol 32)*
Discussion should be available to the Secretary by July 1, 1935

An Investigation of the Permeability of Mass Concrete with Particular Reference to Boulder dam*

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INTRODUCTION

The contract specifications for the mass concrete for Boulder dam contemplated a mix of one barrel of cement per cubic yard and the use of aggregate of 9 in. maximum size. A 1-in. gunite facing was specified for the upstream face and provision was made for enriching the concrete for a depth of about six feet should investigations appear to make this desirable. The permeability investigations reported in this paper were made primarily to furnish data for making final decisions on the type of construction and on the mix or mixes to be used.

Although considerable technical literature on the permeability of laboratory mixes of concrete was available (Ref. 3, 7, 13, 14 and 17) there seemed to be few data which could be applied quantitatively to such problems as the Boulder dam presented, that is, where the concrete contained large aggregate and special cement, and where the maximum head forcing water into the drainage galleries was over 700 ft. Previous tests had all been made on small aggregate concrete for comparative purposes and no formula was offered to extrapolate the data to full scale structures subjected to mass curing conditions. Although the entire series of tests originally planned has not been completed and other tests will be required to answer questions raised, it is believed the data are adequate to support decisions made with respect to Boulder dam.

This paper consists of four parts: first, a brief description of the laboratory experimental apparatus and technique; second, the test results; third, some practical applications of the data; and fourth, an appendix containing a theoretical discussion of the factors controlling the flow of water through concrete and the resultant corrosion.

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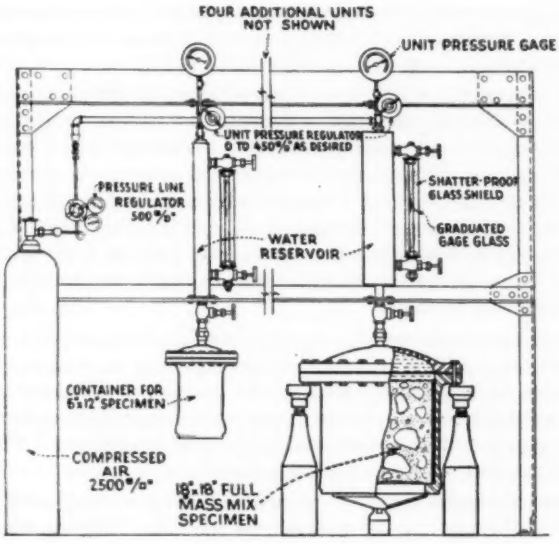

FOUR ADDITIONAL UNITS NOT SHOWN
UNIT PRESSURE GAGE
UNIT PRESSURE REGULATOR 0 TO 450 #/□" AS DESIRED
PRESSURE LINE REGULATOR 500 #/□"
SHATTER-PROOF GLASS SHIELD
GRADUATED GAGE GLASS
WATER RESERVOIR
CONTAINER FOR 6"x12" SPECIMEN
COMPRESSED AIR 2500 #/□"
18"x18" FULL MASS MIX SPECIMEN

FIG. 1—PERMEABILITY APPARATUS
Room temperature 70° F ± 2°—relative humidity 80 percent ± 5 percent

Permeability is probably a more complex property of concrete than strength. Since in these experiments there were but a hundred or so permeability tests of general application, most of the conclusions are tentative and strictly apply only to the data presented. Where examples are given they are largely for illustrative purposes and are not to be taken as accurate solutions of the problem presented.

1. PERMEABILITY APPARATUS, TEST PROGRAM AND EXPERIMENTAL ERRORS

Preliminary considerations indicated that experimental permeability values might be influenced greatly by the grading and size of aggregate, by end effects in the cylinders, by the extent of the chemical reactions at time of test, by temperature and humidity, and by the difficulty of obtaining measurable outflow. The apparatus was, therefore, designed:

(1) For measuring both inflow and outflow;

(2) For testing specimens made with actual field mixes, with aggregate up to 9 in. in size, and with curing which would simulate field conditions;

(3) So that cylinder containers would be long enough to permit investigation of specimen end effect;

(4) For a range of applied water pressures corresponding to a hydrostatic head of 300 to 1,000 ft.

The first requirement fixed the type of testing apparatus, the second fixed the maximum diameter of the test specimen as 18 in., the third established a maximum length of specimen of 24 in. and the fourth necessitated a maximum pressure of about

400 p. s. i. Arrangements were made to permit testing four small specimens and two large specimens simultaneously. Fig. 1 is a diagrammatic representation of the apparatus developed to meet these conditions.

Apparatus. Pressure is supplied by three ordinary oxygen tanks connected to the line in parallel, so that any tank can be disconnected without releasing line pressure. The supply of oxygen in one tank is sufficient for about ten day's run with all units operating. Pressure in the main line is maintained constant by means of a pressure regulator. Branch lines with regulating valves lead to calibrated tanks from which the water is supplied to the test specimens. The tanks were made of 2 in. and 5 in. diameter extra strong steel pipe with welded ends and are fitted with smooth-wall gage glasses. From the bottom of each water tank the line leads through a globe valve to the specimen container.

The specimen containers are of cast nickel steel and are designed to withstand a pressure of 500 p. s. i. The inside diameter provides a ½ in. clearance on all sides of the specimen for the sealing material, and the specimen is supported on a machined ledge. The sealing material used is a commercial pipe-joint asphalt compound. Several other sealing materials were tried but proved less efficient. The upper lid of the specimen container seats on a load ring gasket.

Water passing through the test specimen is caught by a funnel pressed against the bottom flange of the container by wedges from the floor. The top rim of the funnel is provided with a rubber washer but the seal is not air tight. The funnel is soldered to the screw lid of a glass collecting jar.

Materials. Most of the specimens were made with Yosemite Special cement, an average present-day portland cement, with a specific surface as determined by the turbidimeter of 1430 sq. cms. per gram. Other cements used for comparative tests were of laboratory manufacture and the specific surfaces and compound compositions were as follows:

	Yosemite	L-23	L-22a	L-22b	L-24
Specific surface	1430	1620	1300	1730	1280
Per cent C_3S	57	23	31	31	55
Per cent C_2S	24	55	46	46	23
Per cent C_3A	8	6	3	3	6
Per cent C_4AF	6	14	17	17	14

The sand and gravel were the same as used in the construction of Boulder dam, and are well rounded aggregates composed chiefly of limestone and quartzite. The average absorption by weight was about 0.6 per cent in 72 hours at 70°F., giving an apparent porosity of about 1.7 per cent. The moisture content of the room dry aggregate averaged about 0.15 per cent by weight. The grading for the full mass mix, in individual per cent retained on each sieve, was as follows:

	Pan	No. 100	No. 48	No. 28	No. 14	No. 8
Sand	0.6	3.2	10.7	3.9	3.0	4.4
	No. 4	⅜"	¾"	1½"	3" +	
Gravel	7.6	10.9	15.3	17.4	23.0	

In mixes with smaller maximum sizes of aggregate the gravel was graded to give the same relative proportions of each size. All mix proportions and water-cement ratios were by weight.

Ordinary Denver city water was used in the mixing and for all the tests.

Test procedure. Attempt was made to cast permeability specimens for test of a single variable on the same day, and to begin the permeability test at about the end of a 28-day curing period, but in many cases this was not possible. A weight deter-

mination of the fresh mix was made from each batch. After casting, the specimens to be cured adiabatically (mass curing) were sealed in cans, placed in adiabatic curing rooms for a period (usually 28 days) and then stored at 70° F. until time of test. The sealed cans prevented any escape or accretion of moisture during the curing period. The standard cured specimens were stored continuously in a fog room maintained at 70° F. (±2°) and 100 per cent relative humidity, the molds being removed 24 hours after casting.

In preparing a cylinder for test about ⅛-in. of mortar was chipped from the top and bottom surface of the cylinder. It was then wire brushed, placed in the specimen container and sealed in place by means of a bitumastic pipe compound poured hot. To remove foreign matter from the water a layer of cotton was placed on the top surface of the concrete.

Test records and presentation of data. For each test specimen sufficient measurements of cumulative inflow and outflow were made to define adequately the discharge-time relations. The tests were usually continued until the rates of inflow and outflow as determined from the plotted discharge curves were in substantial agreement, generally from 200 to 500 hours after commencement of test. A tangent drawn to the cumulative inflow curve at a point corresponding to 250 hours plus one-half the time to visible outflow determined the discharge rate for computing the permeability coefficient from the equation

$$\frac{Q}{A_e} = K_e \frac{H}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which

Q = rate of discharge, in cubic feet per sec.

A_e = gross area subject to percolation, in sq. ft.

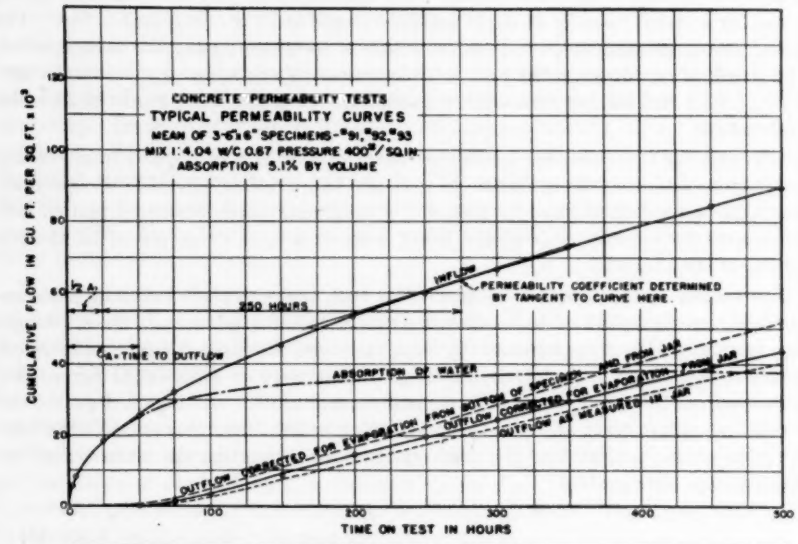
$\frac{H}{L}$ = hydraulic gradient (ratio of head of fluid to percolation length).

K_e = permeability coefficient, or the unit rate of discharge at unit hydraulic gradient, with the dimensions of a velocity, at a temperature of 70°F.

Test program. The permeability data presented were obtained from 138 tests out of a planned program of about 250. In addition to the tests completed about 20 per cent more were started but had to be rejected on account of seal leaks or other causes. The tests have the following approximate groupings but the mixes were so arranged that specimens could be used for more than one purpose:

No. of Specimens	Principal Purpose of Test
12	Preliminary, to test apparatus and establish technique
18	Properties of Boulder dam concrete.
5	Effect of pressure variation.
45	Effect of length of specimen.
12	Effect of end condition of specimen.
10	Effect of size of aggregate.
10	Effect of direction of flow.
5	Effect of cure.
21	Miscellaneous.
138	Total tests.

Errors of tests and reliability of results. In spite of the high pressures used, for the more impermeable concretes the amount of water forced into and through the 6 and 12 in. diameter specimens was in many cases only about 1 and 12 cc. per day respectively and these quantities also represented about the probable error in a single reading of the gage glasses measuring inflow. With such small quantities in question any


CONCRETE PERMEABILITY TESTS
TYPICAL PERMEABILITY CURVES
MEAN OF 3-6"x6" SPECIMENS-"91,"92,"93
MIX 1:4.04 W/C 0.67 PRESSURE 400#/SQ.IN.
ABSORPTION 5.1% BY VOLUME
CUMULATIVE FLOW IN CU. FT. PER SQ.FT. x 10³
INFLOW
PERMEABILITY COEFFICIENT DETERMINED BY TANGENT TO CURVE HERE
250 HOURS
½ A
A = TIME TO OUTFLOW
ABSORPTION OF WATER
OUTFLOW CORRECTED FOR EVAPORATION FROM BOTTOM OF SPECIMEN AND FROM JAR
OUTFLOW CORRECTED FOR EVAPORATION FROM JAR
OUTFLOW AS MEASURED IN JAR
TIME ON TEST IN HOURS

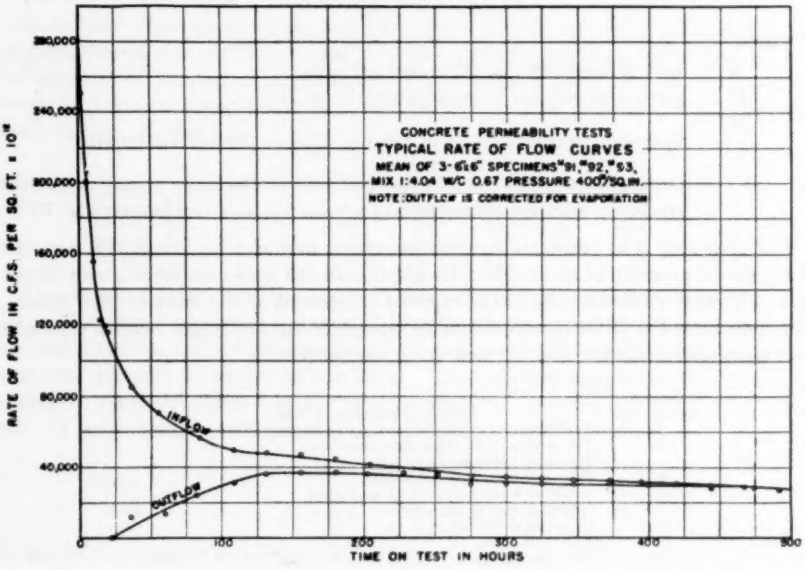

CONCRETE PERMEABILITY TESTS
TYPICAL RATE OF FLOW CURVES
MEAN OF 3-6"x6" SPECIMENS"91,"92,"93,
MIX 1:4.04 W/C 0.67 PRESSURE 400#/SQ.IN.
NOTE: OUTFLOW IS CORRECTED FOR EVAPORATION
RATE OF FLOW IN C.F.S. PER SQ. FT. x 10⁸
INFLOW
OUTFLOW
TIME ON TEST IN HOURS

FIG. 2

FIG. 3

leakage in the water system was a source of appreciable error since 24 drops of water are about equal to 1 cc. Small leakages were difficult to detect in spite of the 80 per cent humidity maintained in the permeability test room. Evaporation also introduced complications in the measurement of specimen outflow, due to the bottom surface of the test specimen being wet and thus subject to some extent to evaporation in the interior of the funnel. Therefore, although inflow and outflow measurements after correction showed approximate agreement, *inflow measurements were always used in computing permeability coefficients.*

Detailed study of the probable errors involved in the values of the coefficients indicated that the test results were subject to a probable error for a single specimen of about ±25 per cent. However, this variation is apparently no greater than other experimenters have found and is characteristic of concrete permeability tests. A probable error of this magnitude should nevertheless be kept in mind in gauging the validity of the relationships derived from a small number of test specimens.

2. TEST RESULTS

Typical tests. As a typical test, Fig. 2 shows the mean curves of total inflow, outflow and absorption (retained inflow), obtained by testing three 6 x 6-in. mortar cylinders. The first outflow was observed about fifty hours from the beginning of the test as evidenced by damp spots on the bottom, which spread until at 150 hours the entire lower surfaces of the specimens were covered with drops of water, each of which appeared centered at the sides of exposed pieces of aggregate. Initially the rate of inflow is high but decreases rapidly with time, while the outflow rate is first low and then increases until the rate (including evaporation) approximates that of the inflow. Before outflow takes place the rate of inflow is given approximately by the equation $\frac{Q}{A} = \frac{C}{T^{0.5}}$. However, since the coefficient C can have many values for the same specimen depending on the free moisture content of the pores immediately preceding the test a determination of the rate of flow into the specimen before uniform outflow is established is not characteristic of the true permeability of a given concrete.

Permeability coefficients. The measure of permeability has been defined as the unit *rate* at which a fluid is discharged from a material under unit hydraulic gradient. As may be seen from Fig. 3, on which the rates of inflow and outflow taken from Fig. 2 are plotted against time, the rate for concrete is a continuous variable. Ideally the rate should be obtained at a time long enough after outflow to eliminate the influence of absorption and any unequal curing effect arising from a difference in the time required to reach outflow. Practical considerations limit the time available for test and force a compromise. Since in most of the bureau's tests outflow occurred within 400 hours, experi-

mentally the permeability coefficient was obtained by writing equation (1) as $K_c = \frac{Q}{A_c}\frac{L}{H}$ and, as previously stated, determining Q from the tangent of the curve of total inflow plotted against time at a time equal to 250 hours plus one-half the time to outflow. (See Fig. 2.)

Aside from the information given concerning the direct loss of water, permeability studies provided information on the pore structure of the concrete and on the effect of corrosion by percolating water. It has been found that the percentage volume of mixing water in concrete is a close approximation of the per cent porosity of the hardened material, and hence, according to the law of homogeneous mixtures, of the per cent of the gross concrete area open to percolation, assuming the aggregate is impermeable. Thus K_v, equal to K_c divided by the absolute volume of mixing water, is directly proportional to the mean velocity of the water passing through the concrete. In the appendix it is shown that the velocity is very sensitive to changes in pore structure, so that studies of the different values of K_v best show the influence of the water-cement ratio and size of aggregate on the internal formation of pores.

Corrosion is a function of the water discharged per pound of cement, therefore, K_c per pound of cement is a direct measure of the relative life of concrete, assuming that percolating water is the only corroding influence.

Finally K_p, equal to K_c divided by the absolute volume of paste, is a convenient unit for comparing the results ofpast experiments with those of the present tests. Comparative values of K_p at a constant watercement ratio are directly proportional to the corrosion of concrete.

Effect of varying pressure. Before discussing the factors controlling permeability in terms of the permeability coefficients, the experimental verification of the equation $\frac{Q}{A_c} = K_c\frac{H}{L}$ is required. The direct dependence of the rated discharge on the head is supported by the results of the two permeability tests shown on Fig. 4 in each of which the pressure was varied during the course of the experiment. The test specimens were first subjected to a high water pressure for a time sufficient to stabilize the outflow. The pressure was then reduced and kept constant long enough to define the characteristics of the new discharge-time relationship and then another pressure reduction was made. By plotting the results on logarithmic paper, it was possible by extrapolation to estimate the rate of flow at the end of the entire test period for each of the pressures used. Thus the actual rate of in-

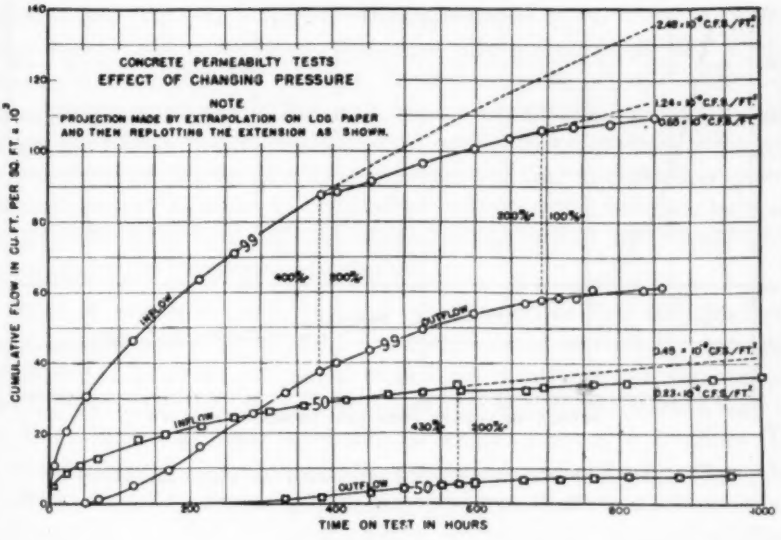


FIG. 4

flow measured at the end of the test for the lowest pressure can be compared with the rates which would have obtained had the higher pressures been maintained. This method eliminates the uncertainties involved in using different specimens with different pressures. Expressing the pressures and rates of discharge in percent of the values for the highest pressures gives the following:

Specimen No.	Pressure	Rate of Discharge
99	100%	100%
	50%	50%
	25%	26%
50	100%	100%
	46%	50%

Combined with the extensive evidence of other experimenters in testing materials other than concrete but of similar pore structures, the direct dependence of permeability on pressure may be accepted.

Effect of curing. It was impractical to test specimens so as to have equal lengths of curing periods. As a result, before the assumed direct dependence of permeability on length could be demonstrated, it was necessary to evaluate the influence of time of cure on the coefficient. The data available are from 12 series of tests, each series consisting of specimens of the same length and mix but tested for permeability at

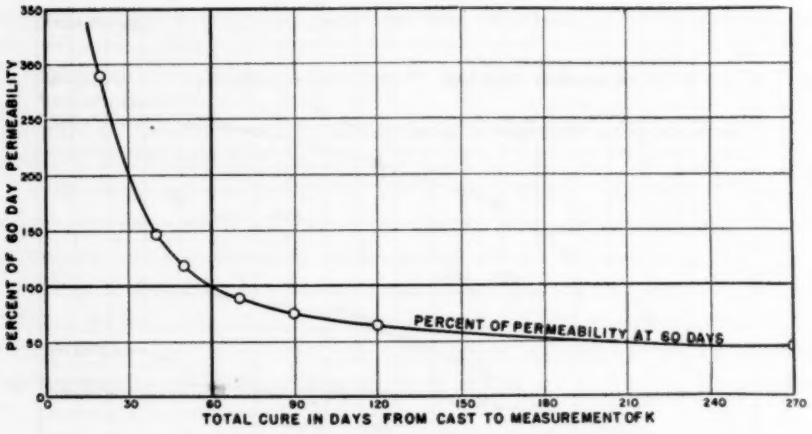


FIG. 5—EFFECT OF LENGTH OF CURE ON PERMEABILITY

different total times of cure. No series covered the total period investigated. In reducing the data the permeabilities for the specimens of each series were plotted against time in terms of the per cent of the mean coefficient for each series and a curve was drawn through the plotted points. This gave 12 overlapping curves, each representing the variation of permeability with time for the particular period covered by a given series. The derived curve (Fig. 5) was obtained by using the permeability at 60 days from time of casting as a standard and then successively computing the percentage change in permeability for small time intervals from this point, using the weighted average of the per cent changes as given by each of the test series applicable. Although the experimental data were widely scattered, due partly to the method of analysis and to the relatively small number of specimens available in most of the test series, the derived data gave a smooth curve. Since most of the data presented in this report were for specimens falling between 40 and 100 total days of cure, the corrections required to bring them to a common basis were not large, and adjustments in the correction curve would not have greatly altered the results.

Mention should be made of the difference between mass and standard curing found in the few tests which were run under comparable conditions. These tests indicated that at 60 days, specimens mass cured in sealed containers under rising temperatures from 40 to 70°F. for 28 days and thereafter at a constant temperature of 70°F. were

about 50 per cent less permeable than specimens cured under standard procedure for the same time. For mass curing starting at 100° F. the specimens were 25 to 50 per cent more permeable. The direction of these results is not out of line with other tests by the bureau for comparing the effects of mass and standard curing on compressive strength. However, since the number of tests were too few to definitely prove these points and in order to bring all experiments to a common basis as far as possible, the results were corrected for cure in accordance with the single curve previously mentioned.

Effect of specimen length and end condition. For flow through concrete in which the percolation distance is long so that the internal resistance is large as compared with local resistances at the ends it seems axiomatic that the discharge will vary inversely with the length. On the other hand, with specimens of lengths from 5 to 20 times the maximum size of aggregate, end conditions are likely to increase the resistance to flow. For all points in a specimen closer to a surface than one-half the diameter of the maximum aggregate the physical makeup of the concrete is not representative of the average concrete and is higher in cement content. As the boundaries of a specimen are approached the concrete grades from a material containing the maximum size aggregate to a mortar and finally to a cement paste. In the appendix it is shown that the pore structure is not only dependent on the water-cement ratio but also varies greatly with the grading of the aggregate and the paste content. It, therefore, seems logical to anticipate a decrease in permeability for the concrete near the boundaries but it is evident that this boundary effect becomes less with respect to unit permeability the larger and longer the cylinders and the smaller the maximum size aggregate.

There is also an effect at the exposed surfaces due to carbonation of the cement during the curing period. Tests on specimens which had been stored for six months showed that carbonation had reached a depth of $\frac{1}{8}$ in. In an endeavor to eliminate this surface effect specimens were chipped top and bottom until the aggregate was exposed. However, it seems possible that this chipping in itself tends to close the pores, being similar in effect to closing the pores of a leaky casting by hammering. Fancher and Lewis (6) found, in experimenting on the permeability of rock, that chipping or sawing was ineffective in eliminating an end effect of substantial magnitude in the 2 in. specimens tested. If end effects existed in concrete, it was thought that they might mask the effects of certain other variables, especially in

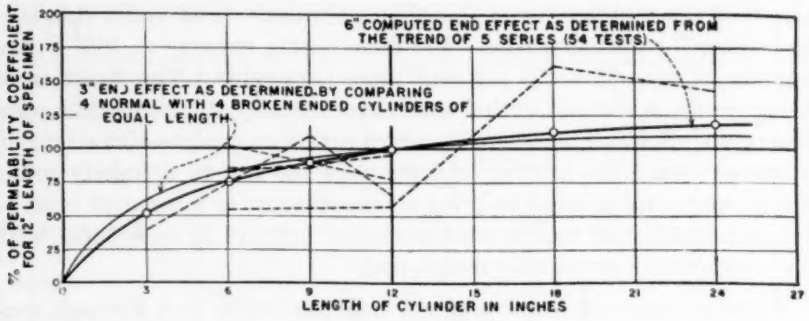

6" COMPUTED END EFFECT AS DETERMINED FROM THE TREND OF 5 SERIES (54 TESTS)
3" END EFFECT AS DETERMINED BY COMPARING 4 NORMAL WITH 4 BROKEN ENDED CYLINDERS OF EQUAL LENGTH
% OF PERMEABILITY COEFFICIENT FOR 12" LENGTH OF SPECIMEN
LENGTH OF CYLINDER IN INCHES

FIG. 6—VARIATION OF PERMEABILITY COEFFICIENT WITH LENGTH OF TEST SPECIMEN

short specimens. The experiments which follow were designed to throw light on this matter.

Two means were available for determining end effect. The first consisted in testing cylinders of similar mix but of increasing length and comparing the computed permeability coefficients, the values of which should approach constancy as the length increases. The second was to compare normal specimens with other specimens of the same makeup and length but obtained by breaking sections out of longer cylinders so as to have freshly exposed ends. Both methods were tried. In making use of the first method, tests were made on specimens from 3 to 24 in. in length but it was necessary to use higher water-cement ratios for the longer specimens in order to obtain satisfactory outflows in a reasonable time.

Using coefficients corrected for cure, five series of tests, each made of the same mix but of different lengths, were available to determine the effect of length on the permeability coefficients. As in the tests used for evaluating cure no series covered the entire range of the variable tested, and the series were of unequal value in respect to the number of individual tests included. Assuming a 12 in. test specimen as a standard, a curve (Fig. 6) showing the effect of length on the permeability coefficient was derived in the same manner as for determining the effect of curing. To show the variation of the experimental data from the final curve, the results for the several test series were superimposed in their respective positions. If, in line with previous reasoning, the failure to secure discharge inversely proportional to length is due to end effect and if such effect is proportional to the true permeability of the concrete in the specimen, then the derived experimental

curve indicates that the two end resistances are equal to that offered by an additional 6 in. length of the interior concrete. For example, according to this curve a 2 in. normally prepared specimen will pass the same quantity of water as would an 8 in. specimen broken from the middle of a longer specimen, and a 24 in. normal specimen the same as a 30 in. one with freshly broken ends.

To check this conclusion directly tests were made of 3 pairs of 6 x 6 in. specimens of which one specimen of each pair was normally prepared and the other was broken out of a 12 in. specimen. The results averaged fifty per cent more water through the specimens broken out of the longer lengths, giving an indicated hydraulic resistance for the two ends equivalent to 3 in. of the concrete tested. The computed curve for a 3 in. end effect is also plotted on Fig. 6. Although it does not seem reasonable that end effect should be independent of the mix, the curve indicating a 6 in. end effect was used in correcting all coefficients to a common 12 in. length.

Variation of permeability coefficients with water-cement ratio and maximum size of aggregate. The principal results of the permeability test program completed to date are shown on Fig. 7, in which the permeability coefficients with respect to voids (K_v) are plotted against the water-cement ratio using logarithmic scales. It is at once evident that water-cement ratio is not the only factor controlling the velocity in the pores but that the type of mix also has an influence of comparable importance. For the mixes tested, at a given water-cement ratio the maximum size of aggregate seemed to be the only variable which gave a consistent relationship with respect to permeability. In Fig. 7 curves have been fitted to points obtained from mixes with equal maximum size of aggregate and it may be seen that the curves express the data in a satisfactory manner. In the appendix it is indicated that settlement between and beneath aggregate particles probably produces relatively large void spaces which have negligible hydraulic resistance, and this possibly is the reason for the increase in velocity found with increase in size of aggregate. For a more complete visualization of the relation between the various permeability coefficients, water-cement ratio and maximum size of aggregate, reference may be made to Fig. 8, in which the curves are reproduced in terms of K_c, K_p and K_v for direct comparison within the practical range of water-cement ratios.

The smaller graph of Fig. 7 is designed to show the effect on permeability K_v which the use of extremely high water-cement ratios beyond the range of practice involves and to give some indication of the effects of substituting an inert material of equivalent fineness for

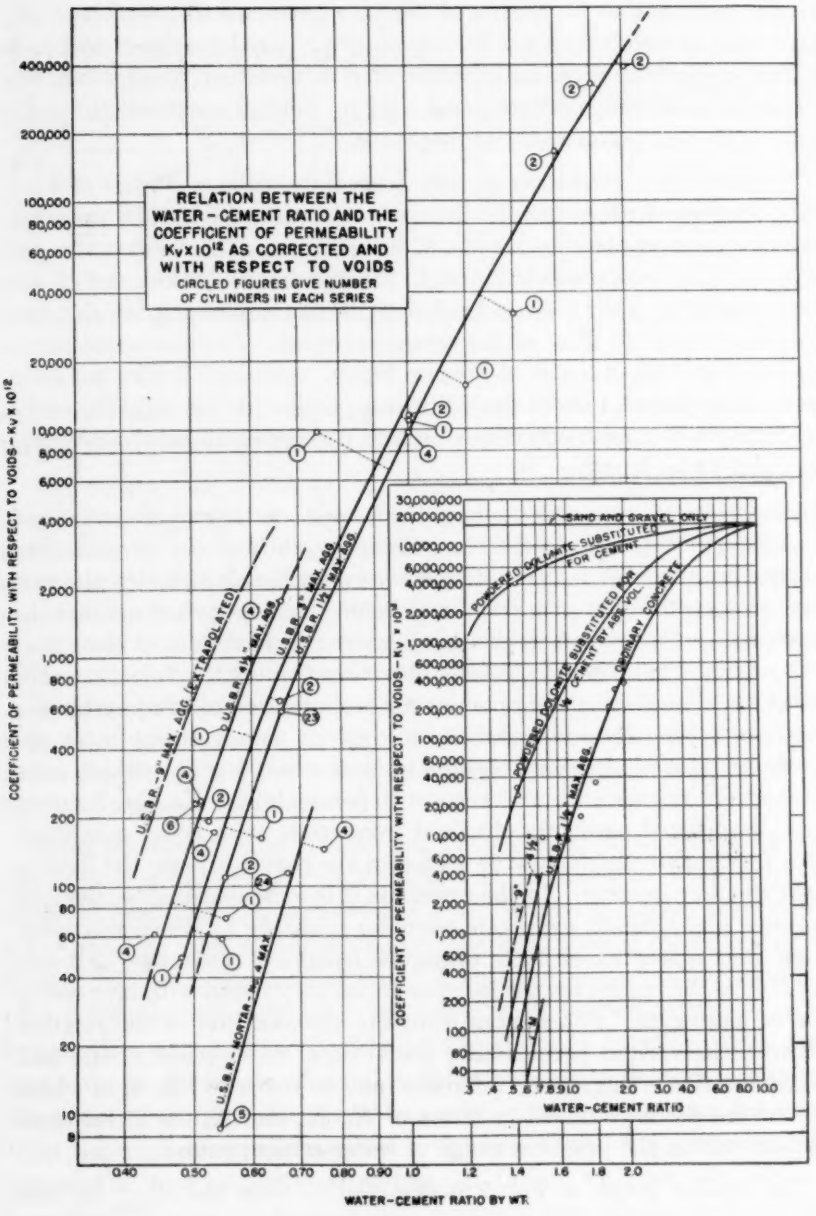

RELATION BETWEEN THE WATER - CEMENT RATIO AND THE COEFFICIENT OF PERMEABILITY Kv x 10¹² AS CORRECTED AND WITH RESPECT TO VOIDS
CIRCLED FIGURES GIVE NUMBER OF CYLINDERS IN EACH SERIES
COEFFICIENT OF PERMEABILITY WITH RESPECT TO VOIDS - Kv x 10¹²
WATER-CEMENT RATIO BY WT.
U.S.B.R. - 3" MAX AGG. (EXTRAPOLATED)
U.S.B.R. 6" MAX AGG.
U.S.B.R. 3" MAX AGG.
U.S.B.R. 1½" MAX AGG.
U.S.B.R. - MORTAR - NO. 4 MAX.
SAND AND GRAVEL ONLY
POWDERED OPAL SUBSTITUTED FOR CEMENT
POWDERED OPALINE SUBSTITUTED FOR CEMENT BY ABS. VOL.
ORDINARY CONCRETE
U.S.B.R. 1½" MAX AGG.
6"
3"
1½"
¾"
WATER-CEMENT RATIO

FIG. 7

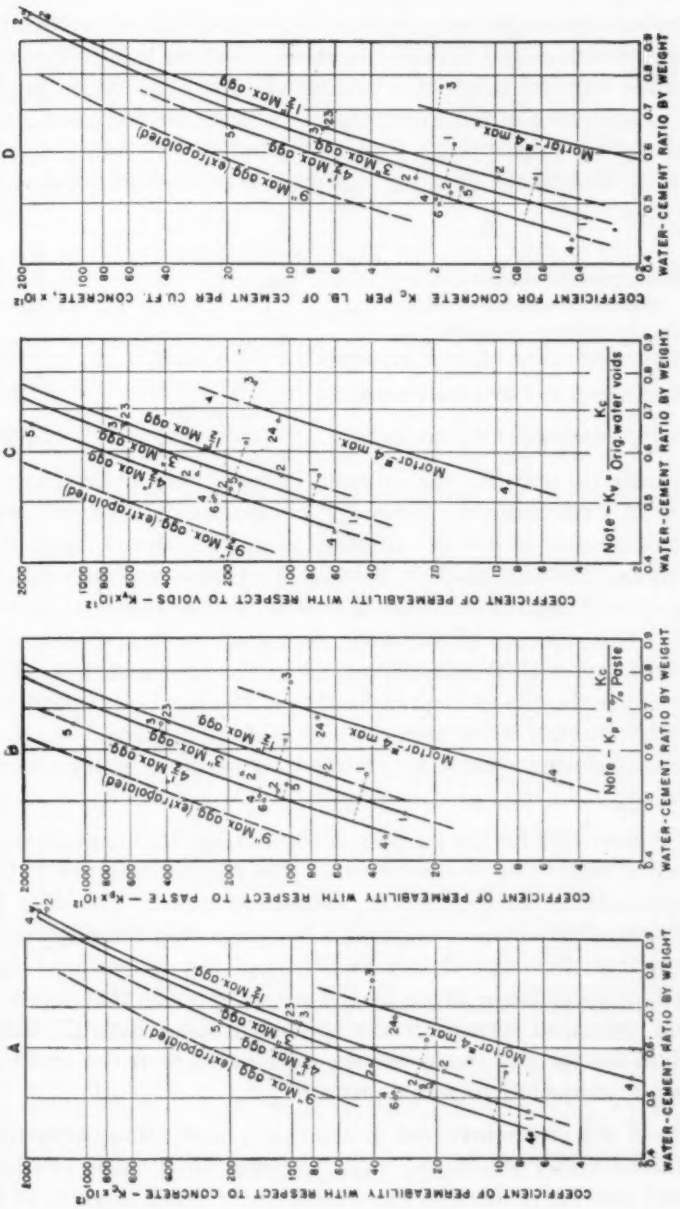

A
B
C
D
WATER-CEMENT RATIO BY WEIGHT
Mortar "a" max.
9" Max. agg. (extrapolated)
4½" Max. agg.
3" Max. agg.
1½" Max. agg.
Note – K_p = $\frac{K_c}{\% \text{ Paste}}$
Note – K_v = $\frac{K_c}{\text{Orig. water voids}}$
COEFFICIENT OF PERMEABILITY WITH RESPECT TO CONCRETE – $K_c \times 10^{12}$
COEFFICIENT OF PERMEABILITY WITH RESPECT TO PASTE – $K_p \times 10^{12}$
COEFFICIENT OF PERMEABILITY WITH RESPECT TO VOIDS – $K_v \times 10^{12}$
COEFFICIENT FOR CONCRETE K_c PER LB. OF CEMENT PER CU. FT. CONCRETE, $\times 10^{12}$

FIG. 8—COEFFICIENTS OF PERMEABILITY EXPRESSED IN VARIOUS UNITS

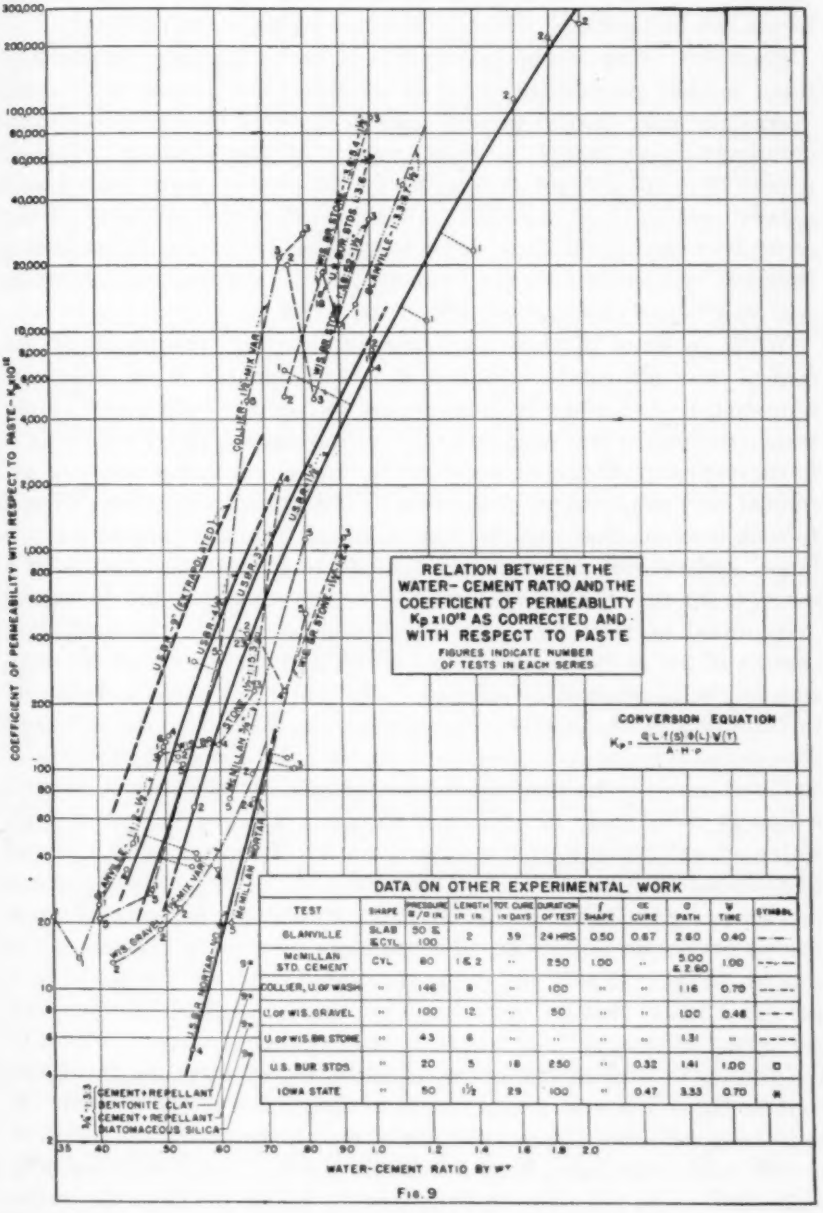
part of the cement by absolute volume. In two of the test specimens the material proportions of a regular mix with 0.53 water-cement ratio were duplicated except that powdered dolomitic sand of approximately the same fineness as the cement was substituted for half of the cement (by absolute volume) in one case and for all of the cement in the other. In a third specimen of the same mix the cement was left out entirely and nothing was substituted. From the permeability curves drawn on the basis of these scant data the following relationships are derived:

	W/C by Weight	K_p
(a) Inert powder substituted for all of the cement by absolute volume	0.5 nominal	4,000,000
Cement mixture consisting of:		
(b) 100% cement and 0% inert material	0.5 actual	40
(c) 50% cement and 50% inert material	0.5 nominal 1.0 actual	20,000
(d) 100% cement and 0% inert material	1.0 actual	10,000

Comparing (a) and (b), the enormous effect of the hydration of the cement is at once noticed. Secondly, comparing (c) and (d), with a water-cement ratio of 1.0 the addition of inert material equal to the cement in amount and fineness, keeping the actual water-cement ratio constant, would tend to increase the permeability rather than to decrease it. Although not enough tests were made to be confident of this result, combined with reasons advanced in the appendix it is believed that the comparison is at least indicative of the need of caution in the use of such materials where permeability is of importance, even though a low cement content might be justified on account of low strength requirement.

Fig. 9 is presented for the purpose of comparing other experiments on permeability with those of the bureau. The results are given in terms of the permeability coefficient with respect to paste. The data from various sources were made comparable by computing the permeability coefficients from the original data and by applying corrections for cure and length of specimen as given by the curves given in this paper. An additional estimated correction was applied to Glanville's (7) data to take care of the form of the specimen, since in his tests the entire section was not subjected to water pressure

In spite of the errors involved in arbitrarily correcting permeability data obtained under conditions much different from those existing in the present tests, it is thought the results are of much interest as they show a general agreement in the permeability properties of concrete as determined under widely different test procedures. It appears that


COEFFICIENT OF PERMEABILITY WITH RESPECT TO PASTE - $K_p \times 10^{12}$
WATER-CEMENT RATIO BY WT
RELATION BETWEEN THE WATER- CEMENT RATIO AND THE COEFFICIENT OF PERMEABILITY $K_p \times 10^{12}$ AS CORRECTED AND WITH RESPECT TO PASTE
FIGURES INDICATE NUMBER OF TESTS IN EACH SERIES
CONVERSION EQUATION
$K_p = \frac{q\,L\,f(s)\,\phi(L)\,\psi(T)}{A\,H\,p}$
DATA ON OTHER EXPERIMENTAL WORK
CEMENT + REPELLANT
BENTONITE CLAY
CEMENT + REPELLANT
DIATOMACEOUS SILICA

TEST	SHAPE	PRESSURE IN LB/SQ IN	LENGTH IN IN.	TOT CURE IN DAYS	DURATION OF TEST	f SHAPE	CK CURE	Φ PATH	Ψ TIME	SYMBOL
GLANVILLE	SLAB & CYL	50 & 100	2	3.9	24 HRS	0.50	0.67	2.60	0.40	— —
McMILLAN STD. CEMENT	CYL.	80	1 & 2	"	250	1.00	"	5.00 & 2.60	1.00	— · — · —
COLLIER, U. OF WASH.	"	146	8	"	100	"	"	1.16	0.70	— — —
U. OF WIS. GRAVEL	"	100	12	"	50	"	"	1.00	0.48	— — —
U. OF WIS. BR. STONE	"	43	6	"	"	"	"	1.31	"	— — — —
U.S. BUR. STDS.	"	20	5	18	250	"	0.32	1.41	1.00	□
IOWA STATE	"	50	1½	29	100	"	0.47	3.33	0.70	■

FIG. 9

FIG. 9

permeability, after allowing for its inherent variability, is as definite a property of concrete as strength and that with a properly standardized test it can be predicted within reasonable limits.

Corrosion. One of the principal reasons for making permeability tests, is that percolating water is corrosive to concrete in varying degree and may in time remove enough material in solution to cause deterioration or complete disintegration of the concrete. Recent papers (1, 8, 16) give solution curves for different cements treated with natural and artificial waters. It is shown that the solubility of the cement at any given time depends principally on the then-existing chemical composition of the cement and on the existing ionization (*pH* value) and chemical content of the water.

While in some of these experiments complete dissolution of the cement was ultimately obtained it is questionable if the tests are representative of what actually occurs in concrete. In many of the tests neat cement was used and the test was made either by shaking it in successive quantities of water, or by finely grinding portions of set cement and subjecting it to continuous filtration. Objections offered to such tests are that with the cement in the form of a powder a much larger surface area of cement is exposed to attack than occurs in a concrete specimen, that the grinding of set cement tends to expose unhydrated portions to water action and to remove the protective coating of the various compounds formed, and that in an actual structure any salts removed by solution at the water face may be deposited in the pores of the concrete downstream so that the pores clog and percolation is reduced or ceases. To obviate the above objections, and to investigate the leaching action and strength loss in concrete, several cylinders were made of standard aggregate grading but using high water-cement ratios so that a large quantity of water could be passed to accelerate the dissolution. The results of the dissolution tests are plotted to logarithmic scale on Fig. 10. As abscissa is the total water passed per pound of cement. The ordinate for the top diagram is the concentration of lime (CaO) in the percolated water in "pounds of lime per cubic foot;" that for the bottom is "total pounds of lime removed per pound of cement." It has been estimated that there is a total of 0.6 pounds of lime in one pound of cement (see lower diagram) and that its entire removal means complete dissolution of the cement. It is to be noted from the curves that although the specimens involved cement contents ranging from 5 to 20 lb. per cu. ft. and the percolating water had mean velocities of from 0.01 to 4.5 ft. per day, the entire series of the bureau's tests as well as those of others using different test methods group together for the first part of the curves with a variation

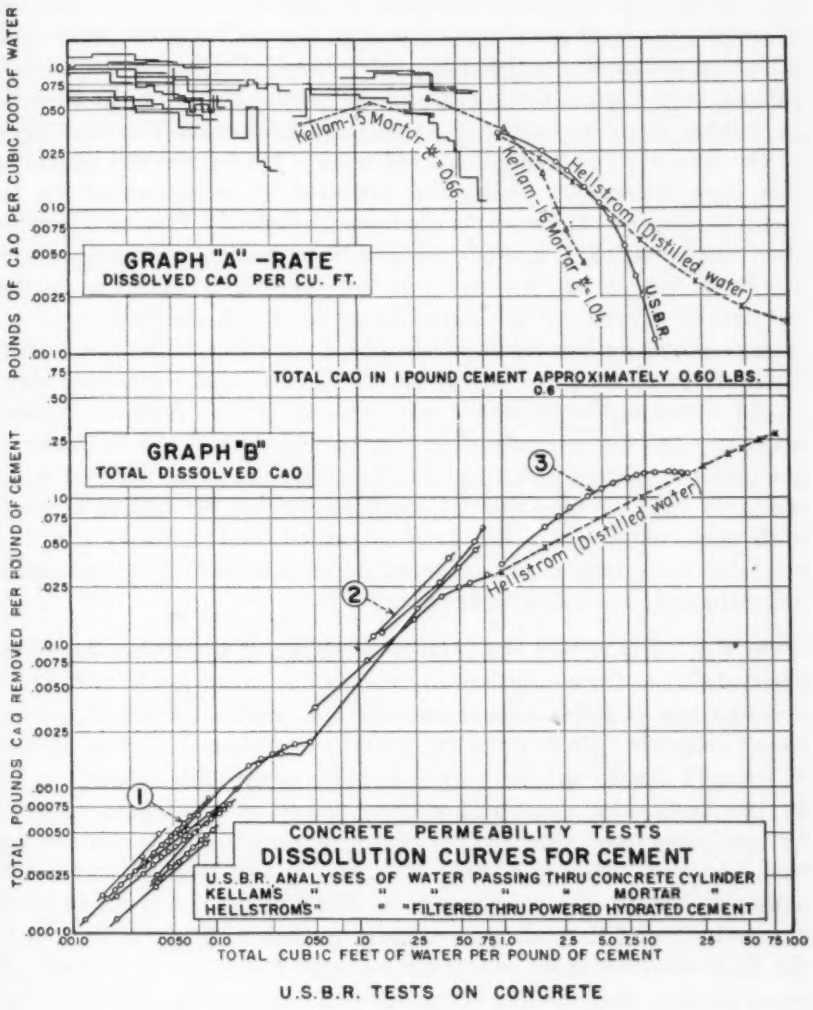

POUNDS OF CaO PER CUBIC FOOT OF WATER
GRAPH "A" -RATE
DISSOLVED CaO PER CU. FT.
Kellam-15 Mortar c = .066
Kellam-16 Mortar c = .104
Hellstrom (Distilled water)
U.S.B.R.
TOTAL CaO IN 1 POUND CEMENT APPROXIMATELY 0.60 LBS.
0.6
GRAPH "B"
TOTAL DISSOLVED CaO
TOTAL POUNDS CaO REMOVED PER POUND OF CEMENT
CONCRETE PERMEABILITY TESTS
DISSOLUTION CURVES FOR CEMENT
U.S.B.R. ANALYSES OF WATER PASSING THRU CONCRETE CYLINDER
KELLAM'S " " " " MORTAR "
HELLSTROM'S " " "FILTERED THRU POWERED HYDRATED CEMENT
TOTAL CUBIC FEET OF WATER PER POUND OF CEMENT
U.S.B.R. TESTS ON CONCRETE

FIG. 10

(1)

Average mix 1:306:543 by weight. Average w/c = 0.66 by weight. Cylinder sizes, 6″ x 6″, 6″ x 12″, 6″ x 18″, 6″ x 24″, 12″ x 12″. Age at test, 28 to 110 days. Pressure, 400 lbs. per sq. in. Velocity per unit void = 0.01 to 0.08 ft. per day.

(2)

Mixes from 1:9 to 1:17 by weight. w/c from 0.75 to 1.2 by weight. Size, 8″ x 4″. Age at test, 3 to 14 days. Pressure, 50 lbs. per sq. in. Velocity per unit void = 0.10 to 0.16 ft. per day.

(3)

Mix 1:19 to 1:29. w/c 1.4 to 2.0 by weight. Size, 6″ x 6″. Pressure, 50 lbs. per sq. in. Velocity per unit void = 0.3 to 4.5 ft. per day. Mean of 7 tests with city water.

of ±50 per cent. Characteristically, as plotted in terms of cubic feet of water percolated per pound of cement, the issuing water starts with a lime concentration approaching saturation. The concentration falls rapidly as the lime freed by hydrolosis is removed. By the time the percolated water amounts to 5 to 10 cu. ft. per pound of cement, most of the lime (0.12 to 0.15 pounds per pound) freed by initial hydrolosis has been removed. Thereafter corrosion is concentrated on the residual silicates. However, in the bureau's tests the Denver city water used was found to be non-corrosive to these residual silicates, for once the early hydrolized lime was removed analysis indicated no further extraction of salts by the percolating water. A comparison of the water analysis for Denver and Colorado River water, with the solubility equilibrium curves given by Baylis (1) indicates that for neither water would corrosion be expected, on account of the relation between alkalinity and ion concentration. On the other hand, the similarity of the bureau's curves with those of Hellstrom (8) using distilled water seems to indicate the essential comparability of the bureau's results with those obtained by the simpler method, and to imply that had corrosive water been used solution of the cement would have continued approximately according to his data.

On Fig. 11 is shown the ratio of strength of specimens tested for permeability to the strength of companion specimens not so tested, and a comparison of water-cement ratio-strength curves for tested and untested cylinders. From these an idea may be obtained of the reduction in strength due to the removal of lime by percolating water. As far as these tests show about a 15 per cent reduction in strength occurred for specimens from which 25 per cent of the original lime content had been removed However since it is well known that an increase in moisture content of a compressive specimen generally tends to lower the strength and since the curves indicate that with but 4 per cent of the lime removed there was a 12 per cent reduction in strength, it seems possible that the loss of lime to an amount of about 25 per cent of the initial content is without great effect on the strength. Taking Tremper's estimate (16) that a 50 per cent lime removal results in complete strength loss, it may be estimated that the security of the concrete depends on keeping the lime content of cement within the limits of 75 and 50 per cent of its original value. Assuming that when the lime content has been reduced to 60 per cent, one-half its strength is gone, this becomes the estimated value of the lime content at which the concrete must be replaced. Based on Hellstrom's data, as plotted on Fig. 10 with water equivalent to distilled water in corrosiveness,

this value will be reached when the total discharge through the concrete is about 35 cu. ft. per pound of cement. It is thought that this discharge represents a minimum for which failure of concrete due to the removal of salts in solution by percolating water can be expected. Snow water in a mountain reservoir may possibly be a type of water for which the minimum figure would be applicable.

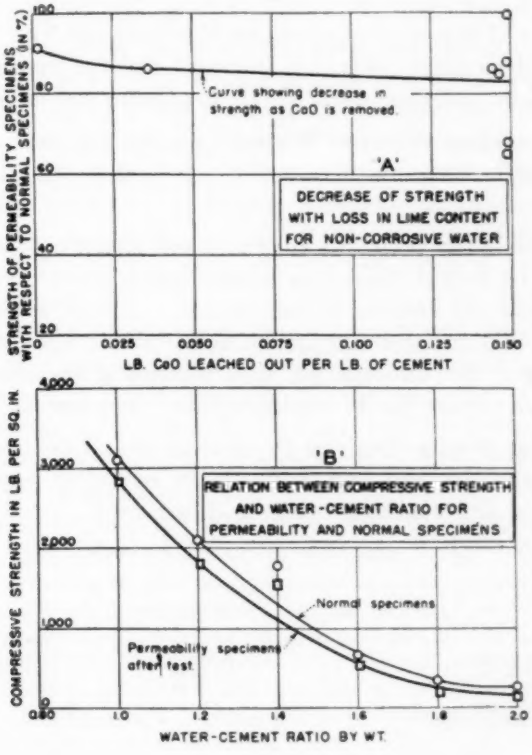


FIG. 11

The question of whether or not the removal of free lime from the pores increases the permeability, or the deposition of salts downstream decreases it has not been answered in these tests. Although in one test 25 per cent of the original lime was removed and the test was continued for four months, no essential change in permeability took place. It is pertinent to remark, however, that while under normal conditions the drop in pressure through concrete is reasonably uniform, if the pores downstream become completely clogged the whole external

pressure will eventually be applied to that face and tend to continue the flow. It seems reasonable to expect that such a condition may lead to physical rupture or spalling of the concrete at the downstream face.

Miscellaneous tests pertaining directly to Boulder dam. A number of the early tests were made with the specific purpose of investigating the permeability characteristics of the proposed Boulder dam mass mix (1:2.45:7.05 by weight, 1.02 bbls. per cu. yd., 9-in. maximum aggregate, and w/c = 0.54±) and the extent to which the characteristics would be altered by changing the type of portland cement, the conditions of curing, and the cement content of the mix. Most of the test specimens were 12 x 12-in. cylinders with 4½ in. maximum size aggregate.

The comparative effects of standard curing and mass curing with different starting temperatures was previously discussed under "Effect of curing."

For the one-barrel per cubic yard mix and an average water-cement ratio of 0.52 by weight, there was no appreciable effect due to changing the composition or fineness of the cement. The influence of type of cement with higher water-cement ratios was not investigated although there is ground for anticipating that increasing the fineness of the cement within certain limits would decrease the permeability.

The average results obtained by increasing and decreasing the cement content of the mass mix (with 4½ in. maximum aggregate), the consistency of the concrete being held constant, are shown in the following tabulation:

Description	Regular Mass Mix	Rich Mix	Lean Mix
Mix proportions by weight	1:2.45:7.05	1:2.04:5.88	1:3.06:8.09
Cement content (bbls./cu. yd.)	1.02	1.20	0.86
Water-cement ratio by weight	0.52	0.44	0.66
Number of test specimens	8	4	5
*Average permeability (K_c x 10^{13})	17	7	230

*Coefficient of permeability in feet per day adjusted to 60 days from time of casting.

It is significant that while an increase in cement content from 1 bbl. to 1.2 bbl. per cu. yd. reduced the permeability only to about two-fifths, a decrease in cement content to 0.86 bbl. increased the permeability about 13 times.

Specimens of the 0.86 bbl. per yard mix with 4½ in. aggregate were also cast and tested in such a way that the direction of water application was at right angles to the direction of casting, to simulate the conditions in the dam. Contrary to expectations, the average permeability coefficient of four out of five such specimens (one test being

rejected on account of exceptionally high discharge) was practically identical with the coefficient for normally tested specimens of the same mix. While further tests are in progress with other mixes and maximum sizes of aggregate, the indications are that direction of water application is not a factor of large consequence.

PRACTICAL APPLICATIONS OF PERMEABILITY TESTS

Concrete permeability tests have three practical fields of application: First, for use in computations in those studies in which it is desired to know the rate of leakage through the walls of a structure such as a concrete pipe line or for determining the effect of concrete in combination with other materials for the purpose of retarding flow as in the facing of an earth dam; secondly, for data as to the amount of dissolved salts removed from the cement of the concrete by percolating waters under any given conditions and hence for giving information as to the probable relative durability and life of concrete as affected by percolating water; and finally, for basic information on the internal pore structure of concrete which has a direct bearing on such things as uplift, absorption, capillarity, resistance to freezing and thawing, shrinkage, effect of waterproofing compounds, etc. The numerical illustrations are given with the realization that the computed results, being based on assumptions which could be improved if the case warranted, have little absolute accuracy. It is believed, never-the-less that they adequately answer the questions raised with respect to the Boulder Dam and that they point out conditions where the permeability value of a concrete may definitely influence a design.

Example 1 (a) Compute the flow through the concrete into the vertical drains near the upstream face of the Boulder dam assuming no shrinkage cracks or other local imperfections; (b) What would be the per cent reduction in flow had the cement content been increased from 1 bbl. per yard to 1.2 bbls. per yard for a six foot layer on the upstream face? (c) What would have been the effect on flow had 1-in. of gunite facing been added? (d) How long will it be before uniform flow takes place through the entire dam?

(a) The dam contains a single row of vertical 8-in. ID drains spaced at 10 ft. centers, giving an average value of head to distance of drain from water face $\frac{(H)}{(L)}$ equal to 10. Concrete has w/c = 0.54 by weight and contains 9-in. cobbles. From the extrapolated curve for 9 in. aggregate of Fig. 8 at w/c = 0.54, K_c = 150, and correcting to values applicable for long specimens by Fig. 6 = 180. For each 10 linear feet of dam there is one drain with an internal perimeter of 2.1 ft. The cross section of concrete through which the water flows to the drain perimeter is, therefore, tapering and the velocity varies. The true path of the water can be determined by the stream-line function. For present purposes the following estimates are made: It is assumed that 1/10 of the water from the front face escapes the drains and goes into the main body of the dam and that the amount of water going into the drain may be computed

by using the average section between the drain and the face. Then $Q = KA\frac{H}{L} =$ discharge per vertical foot of drain =

$$\frac{180}{10^{12}} \times \frac{2.1 + 9}{2} \times 10 = \frac{1.0}{10^{8}} \text{ c.f.s}$$

Since this is a smaller discharge per sq. ft. of drain surface than the measured evaporation per sq. ft. in a room kept at 70° F. and 80 per cent humidity, it would appear that any leakage picked up by the drains will be the result of flow through cracks or construction joints and will not be due to percolation through the homogeneous concrete.

(b) For the 1.2 bbl. per yard mix considered for the front face of the dam the designed water-cement ratio was 0.47. From Fig. 8 the permeability coefficient would be reduced from 150 to 50, and one foot of concrete with a 1.2 bbl. mix would be equivalent in its resistance to flow to three feet of the concrete used. Since near the base of the dam the percolation distance is about 100 feet, the use of six feet of the richer mix for a facing would have reduced the computed flow by

$$1 - \frac{100}{100 - 6 + (3 \times 6)} = 0.11, \text{ or about 11 per cent.}$$

(c) An estimate of the effect of 1-in. of gunite facing can be made by the same methods used in the preceding example. Since, however, percolation through the plain concrete of the Boulder dam is so low that all the issuing water evaporates, no further consideration of the use of gunite is required.

(d) Of considerable interest from the point of view of measuring the hydrostatic heads which exist within dams causing uplift is the length of time required for pressure to become steady across a section of the dam, that is, the length of time until downstream outflow takes place. A very rough estimate can be made by computing the total air space within the concrete which must be filled with water before pressure builds up and outflow takes place. The water is assumed to be supplied by percolating through concrete to the center of gravity of the section.

In a large number of permeability tests a comparison was made between the amount of water absorbed by the specimen and the computed air voids in the specimen. Assuming that this space was the sum of the air entrained in mixing plus the voids created by the hydration of the cement, and that the aggregate was impermeable, this ratio, for a range of computed air voids of 7 to 15 per cent of the paste volume averaged 1.2, with a probable error in the individual results of about 20 per cent. This indicates a 3 to 5 per cent absorption by the aggregate. Applying this method to the concrete of the Boulder Dam the air space unfilled with water is 2.4 per cent. At elevation 700 the head is 506 feet and the thickness of the dam 455 feet. The total discharge required to fill voids = 455 x .024 = 11 cu. ft. per square foot of upstream face. Assuming no drains

$$Q = K_c \frac{H}{L} = \frac{180}{10^{12}} \times \frac{506}{227} = \frac{400}{10^{12}} \text{ cu. ft. per sec.}$$

Time to fill voids and produce full uplift $= \frac{11 \times 10^{12}}{400 \times 3.1 \times 10^{7}} = 900$ years. The above computation assumes that there are no drains. If as was assumed in (a) the drains collect 9/10 of the water, the time will be of the order of 10,000 years. As a matter of interest it may be noted that in a dam the ratio $\frac{H}{L}$ remains nearly con-

stant but that the total discharge required to fill the voids with water increases with the depth. For a triangular section with water at the crest, outflow at the top would take place at once. With the actual sections leakage at the top should be the first to be observed and this applies equally to leakage through construction joints or defects.

Example 2. To reduce the lost head and lower the leakage a 6-in. concrete lining is proposed for a 10-ft. ID tunnel two miles long under a 200 ft. head, carrying snow water and located in a volcanic district free of ground water. The lining is to be placed pneumatically using a mix with 2 in. aggregate, a water-cement ratio of 0.75 and with 1 bbl. of cement per cubic yard (14 lb. per cu. ft.). Compute the leakage and the probable life of the concrete based on its deterioration by percolating water. From Fig. 8, $K_e = 350 \times 10^{-12}$ and by Fig. 6 the value applicable to 6-in. of concrete is 0.8 of this or 280×10^{-12}. Then leakage per sq. ft. of tunnel =

$$K_e \frac{H}{L} = \frac{280}{10^{12}} \times \frac{200}{0.5} = \frac{112{,}000}{10^{12}} \text{ c. f. s.}$$

Total leakage from tunnel =

$$\pi DL \times \frac{112{,}000}{10^{12}} = 0.04 \text{ c. f. s., or } 3500 \text{ cu. ft. per day}$$

Leakage through defects and construction joints might be a thousand times this value. According to estimates previously given, if snow water is equivalent to distilled water in its corrosive power a total leakage per sq. ft. of 35 cu. ft. per pound of cement in the lining will necessitate replacement.

$$\text{Time to replacement} = \frac{35 \times 7 \times 10^7}{3.1 \times 10^7 \times 1.12} = 71 \text{ years.}$$

As in other design problems, adequate factors of safety must be provided in computations concerning corrosion. If a tunnel lining is expected to stand without repair for 20 years it is believed that to give adequate service the computed life should not be less than 200, or a factor of safety of 10. Since permeability per pound of cement for a given mix, neglecting difference in end effects, varies inversely as the square of the depth a 10-in. lining would give a life of 190 years. Alternatively with the water-cement ratio reduced to 0.7 and the cement content increased 20 per cent about the same workability and life would be obtained. Still better possibly would be the use of the same mix with the maximum size aggregate limited to ¾-in. thus promoting greater homogeneity in the field concrete, which is believed to be of prime importance.

Perhaps the relationship between permeability, cement content and relative life of concrete due to corrosion by percolating water is brought out even better by Fig. 8-D. On this figure the results of the bureau's tests are plotted in terms of K_e per pound of cement and the coefficients are thus direct measures of the relative durability of different mixes as affected by percolating water.

Example 3. The face slab of an Ambursen dam is two feet thick, reinforced 3-in. from each face and with inclined shear reinforcement. Discuss the situation with respect to permeability and its effect.

It has been found by examination that concrete settles away from the bottom of reinforcing bars placed horizontally, leaving visible space. It has also been found even with bars placed vertically that in a permeability test water follows the reinforcement unless special preparation and cutoffs are used. In comparison with the

hydraulic resistance offered by the concrete itself, the resistance along the bars is negligible. Consequently the water entering the concrete at a surface exposed to percolation tends to flow along the horizontal bars to a stirrup, then down the stirrup and through the concrete at the bottom. Thus though the apparent leakage distance is 2 feet, the effective distance is but 6 in., giving a high ratio of head to leakage distance, and in consequence a high flow takes place across the two 3-in. reinforcement coverings. Since the lower face of the slab is exposed to evaporation salts precipitated from the issuing water tend to close the surface pores of the concrete and to thus transfer the entire water pressure to the concrete cover of the bottom bar. Under such conditions spalling of the concrete is likely to occur. Evidently contrary to what might be supposed adequate concrete cover and spacing are more necessary for the bottom reinforcement than for the top.

TABLE 1—TABULATION OF TYPICAL PERMEABILITY COEFFICIENTS FOR VARYING MATERIALS

Coefficient represents quantity of water in cubic feet per second, per square foot of surface exposed to percolation, passing through one foot of substance with one foot head. $Q = \frac{K_c H}{L}$

Material	$K \times 10^{12}$	Class
Granite specimen	2 — 10	12
Slate specimen	3 — 7	12
CONCRETE and MORTAR, w/c 0.5 — 0.6	1 — 300	12 — 10
Breccia specimen	20 —	11
CONCRETE and MORTAR, w/c 0.6 — 0.7	10 — 650	11 — 10
Calcite specimen	20 — 400	11 — 10
CONCRETE and MORTAR, w/c 0.7 — 0.8	30 — 1,400	11 — 9
Limestone specimen	30 — 50,000	11 — 8
CONCRETE and MORTAR, w/c 0.8 — 1.0	150 — 2,500	10 — 9
Dolomite specimen	200 — 500	10
CONCRETE and MORTAR, w/c 1.2 — 2.0	1,000 — 70,000	9 — 8
Biotite gneiss in place, field test	1,000 — 100,000	9 — 7
Sandstone specimen	7,000 — 500,000	9 — 7
Cores for earth dams	1,000 — 1,000,000	9 — 6
Slate in place, field test	10,000 — 1,000,000	8 — 6
Face brick	100,000 — 1,000,000	7 — 6
CONCRETE, unreinforced canal linings, field test	100,000 — 2,000,000	7 — 6
*Steel sheet piling—junction open 1/1000" with ½" of contact—18" sections	500,000 —	7
*CONCRETE, restrained slabs with ¼ to ½% reinforcing —30° Temp. change	1,000,000 — 5,000,000	6
Water bearing sands	1,000,000,000	3

*Flow through 1 ft. length of crack, 1 ft. deep, $Q = 60{,}000 \frac{H}{L} D^3$

Example 4. Discuss the permeability of concrete in relation with other materials used in controlling the flow of water.

In Table 1 are tabulated for reference the permeability coefficients for many kinds of engineering materials taken from various sources and arranged in order of increasing permeability. The most noticeable feature of the tabulation is the wide range covered not only by all the materials but by concrete itself. Since the permeability unit has such a wide range and can rarely be closely estimated it is suggested that it be classified by the logarithm of its reciprocal which is the method used in defining the *pH* values of water. The right hand column shows the values of this "impermeability" unit with values of from 3 to 12, an increase of one unit meaning a ten times greater "impermeability."

The permeability coefficients other than those for concretes are not to be taken as representative of all material of a given type since they are based on very few tests and for the rock in place, for reinforced concrete, and for piling are rough estimates. However, certain useful conclusions may be drawn. The coefficients for rock speci-

mens were obtained by means of permeability tests similar to those used in testing concrete and on selected specimens free from cracks. It is believed that the granite and limestone tests are representative of the better class of aggregate and falling in classes 11 to 12 as compared to concrete in classes 9 to 11, the table indicates that good aggregate is of the order of 100 times as impermeable as concrete tending to justify the assumption that the aggregate is impermeable compared to the paste.

The coefficients for reinforced concrete and piling were computed by means of the formula given at the bottom of the table, the coefficient in the formula having been estimated from tests giving leakages past closely fitting pistons in steel cylinders. If these coefficients are even approximately of the right order they show that restrained reinforced concrete without expansion joints and sheet piling compare in tightness with one foot of the core material often used for earth dams, and that they serve as a protection against the formation of direct water channels through the dam rather than as impervious diaphragms. Also it is of interest that according to the estimates of the table, rock in place is, due to incipient cracks, far more permeable than plain concrete.

SUMMARY OF RESULTS AND CONCLUSIONS

A—Of General Application

1. Percolation of water through mass concrete follows the normal laws of viscous flow as expressed by an equation of the form $\frac{Q}{A} = K\frac{H}{L}$ in which K is the permeability coefficient or the unit rate of discharge at unit hydraulic gradient. This equation may be used to estimate the flow if the coefficient as obtained in tests on laboratory specimens is properly corrected for specimen end effect.

2. For the concretes tested, the major factors controlling permeability were water-cement ratio and maximum size of aggregate. For water-cement ratio values from 0.45 to 0.80 by weight, the corresponding range in permeability for mixes containing the same maximum size of aggregate was about 1 to 100. For a given water-cement ratio with aggregate from ¼ to 9 in. maximum size, the average range in permeability was about 1 to 30. In both of these comparisons the cement content was also a variable.

3. Analysis of the physical makeup of concrete indicates that percolating water finds passage mainly through:

(a) Inter-sand voids above the settled cement paste, the size of the voids increasing rapidly with water-cement ratios over 0.4 to 0.5 by weight;

(b) Relatively minute voids in the cement paste, the porosity of the paste and the size of the voids depending more on the state of chemical reaction than on the water-cement ratio, and;

(c) Voids underneath the larger aggregates, caused by the settlement of mortar and paste and depending mainly on the size of the aggregate and the water-cement ratio.

4. Increasing the age of the test specimen, without interruption in curing, to the time when the permeability coefficient was determined caused a relative reduction in permeability in the ratio of 3 to 1 between the ages of 20 and 60 days respectively, and in the ratio of 2 to 1 between the ages of 60 and 180 days respectively. The extent to which percolating water may be expected to compensate for interrupted moist curing was not established in the tests completed to date.

5. Due either to the manner of preparing specimens for test or to other causes, there was an end effect which made short specimens less permeable per unit of length than long specimens. The end effect was found to be equivalent to increasing the length of the specimen about 6 inches (or 3 inches for each end).

6. Percolation of water through concrete gradually removes the chemical compounds of the cement through solution. The amount of percolating water required to bring about a given degree of dissolution, dependent on the character of the supply water, is directly proportional to the cement content. On the premise that 25 per cent removal of the original lime content of the cement is accompanied by little strength loss of the concrete, as indicated by the relatively few tests made to date, it is estimated that at least 35 cubic feet, per pound of cement, of water equivalent to distilled water in corrosive properties must percolate before one-half of the strength of the concrete has been sacrificed.

7. The reasonably satisfactory correlation of permeability test data from many sources by means of the permeability coefficient defined in this paper indicates that permeability is a definite property of concrete susceptible of evaluation.

8. Study of the pore structure of concrete indicates the possibility of uplift acting on 85 to 95 per cent of the pore area of the concrete penetrated. However, in large gravity dams the time required to develop uplift through the entire section may be hundreds of years.

B—Of Special Application to Boulder Dam

1. The Boulder dam mass concrete containing 9 in. maximum size aggregate and approximately one barrel of low-heat cement per cubic yard is so impermeable that flow through the concrete proper (excepting cracks and structural imperfections) may be expected to evaporate on reaching the drain pipes or drainage galleries near the upstream face

of the dam. On the basis of this conclusion the use of a gunite coating or a richer concrete at the upstream face was deemed unnecessary.

2. For specimens of the Boulder Dam mass mix, but with 4½ in. maximum size aggregate, the permeability was not appreciably affected by changing the fineness or chemical composition of the cement, or by substituting mass curing for thorough standard curing. Increasing the cement content from 1 bbl. per cu. yd. to 1.2 bbl. reduced the permeability to about one-third; whereas, decreasing the cement content to 0.86 bbl. increased the permeability about 13 times. For the mix with 0.86 bbl. per cu. yd., changing the direction of water application did not alter the measured permeability.

APPENDIX

Theoretical Considerations

Hydraulic laws. The adequate interpretation and application of the results of permeability experiments necessitate an understanding of the formation and distribution of the openings through which a fluid passes and of the laws which govern such flow. To be permeable implies that connected passages exist through the body of a material and that the minimum diameter of these passages are at least several times the molecule diameter of the percolating fluid. For water this minimum diameter is perhaps 0.02 micron (1/125,000").*

Since many materials contain connected voids of at least this size it is not surprising that such a material as steel will pass a gas or that with respect to water such materials as gravel, sand, brick, paper, porcelain filters, hard-pan, clay, concrete and rock are permeable in a decreasing degre . In all of these materials the water passages are very small and it is to be expected that the hydraulic laws for viscous flow apply. The basic expression for such flow is known as D'Arcy's law and states that for constant physical conditions the unit rate of discharge is proportional to the hydraulic gradient or as an equation,

$$\frac{Q}{A_e} = K_e \frac{H}{L} \qquad (1)$$

Many experiments have been made verifying this law and determining the permeability coefficient K_e for various types of materials and fluids.

While the equation as written above gives the discharge per unit of *gross* area, providing K_e is known, it gives no indication of the factors on which K_e depends. Some idea of these may be had by using the similar equation for flow in capillary tubes known as Poiseuille's law,

$$Q = K_1 A_1 D_1^2 \frac{H}{L} \qquad (2)$$

If the subscripts are taken to define the average properties of the pores in a material through which a fluid passes then substituting this value of Q in the first equation and solving for K gives

*Bulkley in his paper "Viscous Flow and Surface Films," (2) shows that the formerly held view that absorbed water at the walls of a capillary tube of sub-microscopic size prevented flow is erroneous or at least that such rigid water is less than 0.01 micron in thickness. Some idea of this dimension may be obtained when it is realized that the limit of vision with a microscope is 0.5 micron and that thereafter capillary passages cannot be seen, though their presence may be inferred.

$$K_e = K_1 \frac{A_1}{A} \frac{L}{L_1} D_1^2 \quad \ldots\ldots (3)$$

It is thus seen that the permeability coefficient depends on K_1, which varies with temperature, viscosity and density of the fluid and upon the cross section and uniformity of the pores; on $\frac{A_1}{A}$, a function of the porosity of the specimen, on $\frac{L}{L_1}$, the ratio of the apparent to the true length of the pore, and on the square of D_1, the effective diameter of the average pore.

Physical makeup of concrete. In this discussion all aggregate will be considered impermeable. As a definite example Table 2 gives the physical characteristics of one of the mass concrete mixes tested for permeability, including for the purposes of visualization certain hypothetical calculations on the size, number and spacing of its component particles. Consider the building up of one cubic foot of concrete by successively imagining each class of material to be added dry to a one cubic foot container. For the mix considered an apparent volume of 0.845 cu. ft. of dry coarse aggregate of average diameter of about ⅓-in. (9,000 microns) with 30 per cent porosity is first added. In this condition with the aggregate particles in point contact, if water were applied under pressure it could circulate freely through the interior and the material would offer a resistance to flow about equal to that of a uniform capillary tube of 1/9-in. diameter, that is, of a diameter about one-third the size of the average particle. (Ref. 6.) If now to the coarse aggregate sand be added of such fineness as to pass freely into the 1/9-in. void spaces (for the case considered the sand has an average diameter 1/15 of the void diameter and 33 per cent porosity) not only will the absolute void volume of the coarse aggregate be reduced by one-third but the new passages formed will approach in size about one-third the average diameter of the sand particles.

By referring to equation (3) it may be seen that the filling up of the voids of the coarse aggregate by material with an average diameter much smaller than the void spaces decreases the permeability coefficient very rapidly since it reduces the porosity $\frac{A_1}{A}$, the length ratio $\frac{L}{L_1}$ and the mean diameter D_1 of the pore.

Considering the quantity of fine material required it would seem that an apparent volume of sand at least equal to the absolute voids in the coarse aggregate would be necessary. However, if the fine material is not uniform in size and if, as in the present case, the largest particles, say ¼-in., are larger than the average void in the coarse aggregate it is evident that in the mixing of the two classes of materials interference will result, leaving relatively large unfilled openings. In order to avoid this condition and to improve workability, sand in excess of the voids is usually added. For the example being used, the apparent volume of sand was 126 per cent of the voids in the coarse aggregate. With the two materials dry mixed together some interference takes place and the result is an apparent volume of 0.983 cu. ft. of combined aggregate with a porosity of 19.0 per cent.

This process of reduction of void space and pore diameter by adding a material whose grains are considerably smaller than the existing pore space could manifestly be carried on as many times as there are grades of materials of sufficient size difference to avoid particle interference. In the present case cement is the next material available and an apparent volume of 0.149 cu. ft. of cement measured dry with a porosity of 51 per cent is used to fill a volume of 0.187 cu. ft., the average particle size of the

TABLE 2—PHYSICAL PROPERTIES OF SPECIMEN 43

Size of specimen 12″ diameter x 12″.
Mix by weight 1:2.45:7.05.
Cement content 1 bbl. per yd.
w/c = 0.53 by weight.
Mass cured 28 days.
Initial temperature 72°.
Age at test 48 days.
Absolute volume water and cement 19.2%
Absolute volume entrained air 1.1%
Absolute volume paste 20.3%
Unit compressive strength of this cylinder 3370 lbs. per sq. inch.

Discharge at 1037 ft. head, Q per day per sq. ft. concrete $\frac{4.9}{10^7}$ cu. ft.

Estimated true velocity $= \frac{Q}{1/3 \text{ vol. voids}} = \frac{1.1}{10^6}$ ft. per day.

	Coarse Aggregate	Fine Aggregate	Combined Aggregate	Cement	Concrete.
1. Maximum size	4½″	¼″	4½″	——	4½″
2. Sp. Gr. at time of pouring	2.70	2.64	2.68	3.23	2.52+
3. Sp. Gr. at time of test	2.70	2.64	2.68	2.84	2.52
4. Unit weight, dry rodded Lbs. / cu. ft.	118	109.5	135	——	155.7
5. Volume of dry material in 1 cu. ft. of concrete, separately measured	0.845	0.316	0.983	0.149	0.983
6. Unit porosity of material	30	33	19.0	51.4	13.09
7. Porosity at time of test%	——	——	——	——	11.50
8. Estimated effective dia. of particles, microns	9000	180	180	1–3	1–3
9. Absolute vol. per cu. ft. concrete	0.59	0.21	0.80	0.07	0.99
10. Estimated total No. of particles per cu. ft. concrete	5600	4.7×10^8	4.7×10^8	1.7×10^{15}	1.7×10^{15}
11. Mean distance between particles of similar kind in concrete, cubical spacing, microns	17,000	330	330	1.4	——

+Apparent specific gravity.

COMPUTED MAKEUP OF PASTE (PER CU. FT.) ABSOLUTE VOLUME

Item	At Time of Pour	At Time of Test U. S. B. R.	Neat Cement Tests McMillan and Yoshioka
1. Cement	0.35	0.46	0.48
2. Free water	0.59	0.42	0.38
3. Air	0.06	0.12	0.14
Total	1.00	1.00	1.00
Ratio $\frac{\text{Air}}{\text{Air + Water}} = R$	0.15	0.22	0.27
Paste porosity	0.67	0.56	0.53

cement being 1/20 to 1/60 of the average pore of the combined aggregate. If no particle interference takes place and the cement packs in the voids between the sand grains with the same density it had when measured dry, each void will be but 80 per cent filled with dry cement. In this condition with respect to percolation the reduction in permeability will evidently not be of the same type as occurred when 126 per cent sand was added to the void spaces of the coarse aggregate, since the spaces between sand grains are not completely filled with the finer material. The percolating water would pass mainly through the upper 20 per cent of the intercommunicating void spaces rather than be forced through the smaller passages between cement grains. It is to be noted that for the dry mix as thus imagined the aggregate particles are in point contact with one another, no shrinkage is possible and the quantity of cement added has little tendency to change the apparent volume of the combined materials, since it is considerably less than the available void space.

Experience has shown that it is impossible to mix concrete without including some entrained air. For the example in question this, as measured, amounts to an absolute volume of 0.011 cu. ft., which may be considered as uniformly dispersed through the mixing water. Adding the aerated water to the imagined mix a total of 0.130 cu. ft. of fluid is forced into the 0.114 cu. ft. of voids existent in the dry mixed material, forcing the particles apart and filling up the one cubic foot container, the ratio of added fluid to existing voids being 1.14. This final addition results in an unstable and expanded condition of the aggregate and the fine particles of cement are in a temporary state of suspension. The aggregate is then said to be floating in the paste. The concrete is now in the same condition as when it comes from the mixer.

Inherent permeability. With the above conception of the physical makeup of the specimen it is seen that what may be called the inherent permeability of the concrete, or the permeability which would obtain providing no chemical changes took place, is largely controlled by the amount and properties of the finest of the combined materials, i. e., the cement and sand. It will thus be a function of the water-cement ratio (w/c), the paste-cement ratio $\left(\frac{w + c}{c}\right)$ and the paste-aggregate ratio $\left[\frac{w + c}{1 - (w + c)}\right]$ expressed in absolute volumes, and of the fineness and grading of the solid materials, as all these parameters have the physical significance of defining different properties of the distribution and size of the spaces in the concrete through which water percolates.

The particles of the concrete just after placement in the forms are not in static equilibrium. Although the aggregate is loosely spoken of as floating in the paste, actually since its specific gravity is greater than that of the fluid paste the condition of a particle resembles that of a steel piston resting on water in a close fitting open cylinder. The weight of the piston forces water upward through the clearance space and the piston settles. Similarly the weight of the aggregate resting on the paste forces water upward against gravity through the void spaces and there is an active hydraulic gradient equivalent to the submerged weight of the aggregate particles. The aggregate is said to settle and the water to gain but from the hydraulic viewpoint a test of the inherent permeability of the mix is being made. For normal mixes the rate at which water appears at the surface of the specimen is a rough indication of the inherent permeability of the material previous to cement hydration. For example, in one of the 36 x 72 in. strength test specimens of the bureau with a 1:3:6 mix containing large aggregate, a total of ⅛-in. of water appeared during the first hour after casting. Assuming this water had appeared at a constant rate, and using as the active head the submerged weight of the material, computation gives $K_c = 2.1 \times 10^{-6}$ ft. per sec. A direct permeability test made on a 4 x 9-in. cylinder with a 1:3.0:5.3 mix, replacing the cement with powdered sand of equivalent fineness, gave $K_c = 0.7 \times 10^{-6}$ ft. per sec. Thus the range of inherent permeability is possibly of the order 10^{-5} to 10^{-7} ft. per sec., the actual value being controlled by the mechanical grading of the mix. It is thought that this conception of an inherent permeability may have application in studies of workability since the ability of a mix to stay together without water gain is probably closely related to the hydraulic phenomenon just described.

Initial shrinkage. Fig. 12 shows some tests made by the bureau on the behavior of equal volumes of cement pastes of different water-cement ratios and cement fine-

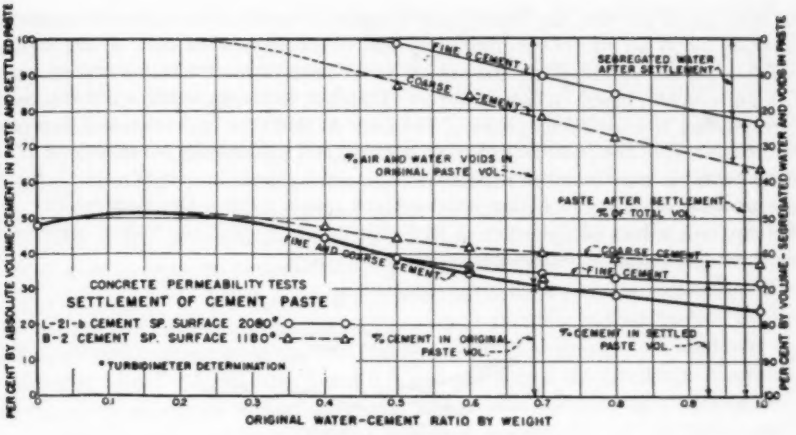

PER CENT BY ABSOLUTE VOLUME-CEMENT IN PASTE AND SETTLED PASTE
PER CENT BY VOLUME - SEGREGATED WATER AND VOIDS IN PASTE
FINE CEMENT
COARSE CEMENT
SEGREGATED WATER AFTER SETTLEMENT
% AIR AND WATER VOIDS IN ORIGINAL PASTE VOL.
PASTE AFTER SETTLEMENT % OF TOTAL VOL.
FINE AND COARSE CEMENT
COARSE CEMENT
FINE CEMENT
CONCRETE PERMEABILITY TESTS
SETTLEMENT OF CEMENT PASTE
L-21-b CEMENT SP. SURFACE 2080*
B-2 CEMENT SP. SURFACE 1180*
*TURBIDIMETER DETERMINATION
% CEMENT IN ORIGINAL PASTE VOL.
% CEMENT IN SETTLED PASTE VOL.
ORIGINAL WATER-CEMENT RATIO BY WEIGHT

FIG. 12

ness placed in test tubes and allowed to settle. The curves at the top of the figure show the amount of settlement, while those at the bottom show the absolute volume of the cement in per cent of the original volume and also in per cent of the volume of paste after settlement. Referring to the finely ground cement L-21-b, the porosity measured dry is estimated at 52 per cent. Mixed at a water-cement ratio of about 0.35 by weight the porosity is the same; at 0.45 the porosity is 58 per cent, which was the highest obtainable without having settlement. With higher water-cement ratios the particles are no longer in static equilibrium and settle, as in the case of the aggregate previously described. Although these cement settlement tests were made in $\frac{3}{8}$-in. glass tubes it is believed that phenomena of a similar nature take place in the interaggregate spaces of freshly poured concrete. Fig. 13 shows a pore between sand grains and by reference to Table 2 it may be seen that being large as compared to the mean cement particle the pore walls have little effect in preventing settlement.

The formation of the internal pore structure of the concrete to the point of set appears to be as follows. Immediately after placement all the particles of the concrete including the cement are to some extent in unstable equilibrium and their weight forces water and fine cement particles to the surface. Gradually the larger particles stabilize, forming a skeleton within which settlement continues. The mortar settlement in turn forces water upwards, some of which comes to rest below the larger pieces of aggregate. Finally between the sand grains, the cement settles to form a structure of 55 to 65 per cent porosity. Of importance is the fact that the voids are no longer homogeneously distributed in the paste but consist of relatively large spaces beneath the aggregate connected with voids of a much smaller order of magnitude distributed in the settled paste.

Hydration period. During hydration water is combined chemically with cement. The exact amount of water combined and the specific gravity of the new product depends upon the particular chemical composition of the cement, the temperature at which the specific gravity is determined and the conditions under which the reaction takes place. Using generalized figures derived from data given by McMillan (11) and Yoshioka (18) for neat cements, in their tests it was found that the solid phase had

increased by 43 per cent, the liquid phase diminished by 63 per cent, and the air phase increased by 20 per cent of the absolute volume of the original cement. At the bottom of Table 2 is shown the effect this has on the volumes of materials in a typical paste with an estimate, based on data given by McMillan, of the change in state of a paste corresponding to a hydration period of 48 days. At that time hydration had increased the air content of the paste from 8 to 14 per cent and reduced the porosity from 67 to 56 per cent. It is pertinent to inquire how this small change in porosity, which would be even less for pastes of higher water-cement ratios, reduces the permeability coefficient from values of the order of 10^{-7}—10^{-5} to 10^{-11}—10^{-9}, the values commonly found for concrete, a one-ten-thousandth reduction.

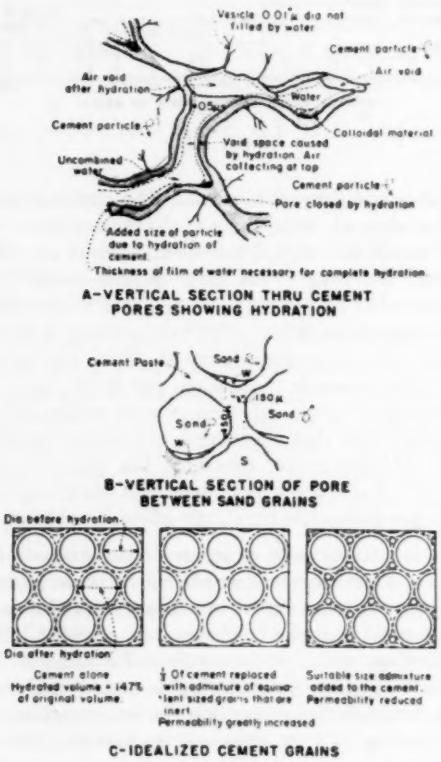

Vesicle 0.01 μ dia not filled by water
Cement particle
Air void
Air void after hydration
Water
0.5 μ
Cement particle
Colloidal material
Void space caused by hydration Air collecting at top
Uncombined water
Cement particle
Pore closed by hydration
Added size of particle due to hydration of cement
Thickness of film of water necessary for complete hydration
A—VERTICAL SECTION THRU CEMENT PORES SHOWING HYDRATION
Cement Paste
Sand
W
150 μ
Sand
Sand
W
S
B—VERTICAL SECTION OF PORE BETWEEN SAND GRAINS
Dia before hydration
Dia after hydration
Cement alone Hydrated volume = 147% of original volume
½ Of cement replaced with admixture of equivalent sized grains that are inert Permeability greatly increased
Suitable size admixture added to the cement. Permeability reduced
C—IDEALIZED CEMENT GRAINS

FIG. 13

According to Baylis (1) and Colony (4) there are indications that cement particles hydrate in place, and therefore under the particular conditions just quoted, the hydrated particle of cement is virtually the original particle increased 13 per cent in diameter. Although this conception is not literally true, it is believed to substantially represent conditions as the particles are not hydrated instantly—the reaction gradually working inward and building a hydrated layer of material around the original

particle. On Fig. 13-C is an idealized diagram with the particles considered as spheres and it may be seen that the effect of hydration is to form small connecting columns of solid material between the particles at their points of contact. As the diagrams are in two dimensions and actually the material is in three dimensions the obstruction caused by hydration is much less than represented. However, in spite of the relatively small change in the total porosity, it is evident that at points of minimum opening a tremendous reduction in diameter of the water passages takes place. As resistance to flow depends on the square of the diameter it appears that this great reduction in diameter and the formation of colloidal material is the cause of the great decrease in permeability observed in hardened cement. Fig. 13-C also illustrates the possible effect of using powdered admixtures with the cement. If these materials are of the same grain size as the cement particles and are inert or increase less in volume, it is apparent that the permeability will be greatly increased over that for cement alone. On the other hand, if a lean mix is used the advantage of avoiding settlement of the paste in inter-aggregate spaces with the consequent formation of large unfilled voids may outweigh the increased permeability of the paste proper The last sketch shows that if the admixture particles are of a size to improve the grading of the cement, small percentages may greatly decrease the permeability. However, this possibility of improvement does not appear to be great since particles of a size to fill the interstices of the cement would be so small that molecular forces would overshadow the effect of gravity and the usual rules of grading would no longer be applicable.

Testing period. Water entering the specimen under the applied head gradually advances, forcing its way through the specimen and pushing ahead of it uncombined water saturated with the soluble sulphates and the lime freed by the chemical reactions. Air tends to be entrapped in the larger void spaces, the air volume contracting as the pressure builds up. The rate of inflow continually decreases according to a complex law depending on the amount of air in the larger void spaces, the amount of water shoved in advance by the moving water front, and the distance the water has moved through the specimen. Outflow first occurs for those water filaments which have had the easiest path through the specimen, resulting in a slowing down in the rate of decrease of the inflow. Finally water flows through the whole cross-section and the percolation rate soon becomes relatively steady, further decrease in the rate being due only to the continued hydration of the cement by the percolating water under pressure.

The velocity of percolation in the usual test is relatively slow, so that solution equilibrium is established between the water and the chemical compounds of the cement. Thus the concentration of salts in the percolated water at any time depends largely on the ratio of the total discharge divided by the pounds of cement in the specimen. Early in the percolation history the readily soluble sulphates and some of the lime freed by hydrolysis are removed by the percolating water. During this period it appears that the percolation rate would increase were it not that continued reaction exerts the contrary effect. Later with the slowing down of the chemical reactions and with water corrosive to the silicates the continuous removal of material enlarges the water passages, the percolation rate increases and the concrete losing strength, begins to disintegrate.

ACKNOWLEDGMENTS

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The authors wish to make special acknowledgment to Assistant Engineers B. L. Pickett and G. A. Samson and Junior Engineer A. F. Johnson for their assistance in compiling, analyzing and plotting the data presented.

BIBLIOGRAPHY

1. Baylis—Trans. A. S. C. E., 1927, p. 790—"Corrosion of Concrete."
2. Bulkley, R.—Bur. St. R. P., Jan. 1931—"Viscous Flow and Surface Films."
3. Collier, Ira L.—Concrete Magazine, Aug. 1928, pp. 35-38—"Permeability of Concrete."
4. Colony—Columbia Univ. Publication—"An Investigation of Constituents and Properties of Cement."
5. Davis—Progress Reports—Oct. 1932, March 1933—"Cement Investigations for the Hoover Dam."
6. Fancher and Lewis—Ind. and Eng. Chem.—Vol. 25, No. 10, Oct. 1933, p. 1140—"Flow of Simple Fluids Through Porous Materials."
7. Glanville, W. H.—Dept. Sci. and Ind. Research, Bld. Res. T. P. No. 3—"Permeability of Portland Cement Concrete."
8. Hellstrom—Structural Eng., May 1933—"Decay and Repair of Concrete and Masonry Dams."
9. Jumper—B. S., R. P. No. 394, Dec. 31, 1931—"Tests of Integral and Surface Waterproofing for Concrete."
10. Kellam, B.—Appendix II, Report of Comm. C-9, A. S. T. M., 1933—"The Action of Water on Concrete."
11. McMillan, F. R.—Eng. News-Record, Vol. 102, 1929, pp. 580, 625, 673, 705, 749—"Basic Principles of Concrete Making."
12. McMillan, F. R.—Letter Nov. 8, 1934 (U. S. B. R.).
13. McMillan and Lyse—Jour. Amer. Concrete Inst., Dec. 1929—"Some Permeability Studies of Concrete."
14. Norton and Pletta—Jour. Amer. Concrete Inst., May 1931—"Permeability of Gravel Concrete."
15. Terzaghi, K.—Eng. News-Record, March 8, 1934, p. 316—"Action of Water Pressure on Fine-Grained Soils."
16. Tremper, B.—A. C. I., 1932, p. 1—"The Effect of Acid Waters on Concrete."
17. Withey—Bul. Univ. Wisc., Eng. Series Bul. 1245, 1923—"Permeability Tests on Broken Stone Concrete."
18. Yoshioka Abstracts, Concrete Institute, 1932, p. 102.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32) Discussion should be available to the Secretary by July 1, 1935*

Placing Concrete by Means of Vibration

Introducing the work of A. C. I. Committee 609, Vibration of Concrete

ALFRED E. LINDAU, CHAIRMAN

Editor's Note—*Committee 609, Vibration of Concrete, was authorized by the Board of Direction and organized under the Chairmanship of Alfred E. Lindau, with H. L. Flodin as Secretary, in the fall of 1934. Its assignment is to prepare a manual on the use of vibration as a means of placing concrete.*

Preliminary work was necessary in exploring field-developed facts from both satisfactory and unsatisfactory experience. Laboratory and field tests had both given strong indications of good results to be obtained through this method. Yet, as its use became more general in the field, it was obvious that optimum results in quality and economy were to be expected only when conditions were controlled to favor the special factors peculiar to satisfactory results from vibration as distinguished from the factors which have governed in other placing methods. The Committee, through a questionnaire sent to many users of vibratory placing, sought the data from which to distinguish the controlling factors of satisfactory work. At the 31st Annual Convention of the Institute Chairman Lindau discussed briefly the preliminary work of the Committee. Ten papers were presented, most of them brief. From this start and from interest and cooperation developed through presenting the substance of its preliminary work for discussion, the Committee hopes to progress toward preparation of its outline of recommended practice. The testimony presented at the convention was in contributions from the following in the order of presentation:

C. M. Hathaway, Engineer of Construction, Illinois Highway Dept., Springfield

F. V. Reagel, Engineer of Materials, Missouri State Highway Dept., Jefferson City

W. R. Johnson, Concrete Technician, Wheeler Dam, Tennessee Valley Authority, Town Creek, Ala.

M. O. Withey, Professor Mechanics, University of Wisconsin, Madison

Thos. E. Stanton, Jr., Materials and Research Engineer, California Division of Highways, Sacramento

Lewis H. Tuthill, Testing Engineer, The Metropolitan Water District of Southern California, Banning, Calif.

F. H. Jackson, Senior Engineer of Tests, Bureau of Public Roads, Washington

BEN MOREELL, Lieutenant Commander Bureau of Yards and Docks, Navy Dept., Washington

SAM COMESS, Assistant Engineer, Rock Island District, U. S. Corps of Engineers, Rock Island, Ill.

T. C. POWERS, Assistant to the Director of Research, Portland Cement Association, Chicago

INTRODUCTION, BY THE CHAIRMAN

It would be difficult to conceive that in the time since the organization of the committee we could report on such a subject. Vibration has only recently come into general use. The nucleus of the committee in Chicago, after looking over the situation pretty thoroughly, decided that the only thing to be done, practically, would be to survey the field and try to bring our information to date. There has been little laboratory research. Vibration has been put to work in the field to see what could be done with it, and the papers that we have this morning are a record, as nearly as we can get it over a wide range of experience and a considerable geographical area, of how vibration has worked practically. The committee has arrived at no analyzed concensus of opinion. For the present we give you the data and the opinions of the authors of these papers; the committee will probably go ahead to find the answers to questions still open.

There is no desire on the part of any one connected with this work to try to make it look complicated. However, things that appear to be simple and probably are simple in essence, require some complicated investigation to prove that they are simple. We are trying to get at the fundamental principles involved as well as the practical application of those principles. The committee asked such questions as these:

"How is the type, power and size of vibrator selected for a particular kind of job?"

"What volume of concrete can be effectively handled per vibrator?"

"What is the most desirable frequency of vibration?"

"Have you observed any effect of changing amplitude with constant frequency or of changing frequency with constant amplitude?" (That affects the manufacturer, but the information must come from the user.)

"To what extent is form pressure increased?" (The contractor wants to know that.)

"What kinds of mixes have been successfully placed?" (The engineer wants to know that.)

"What characteristics of mix may be changed from those required for hand placing?"

"How should concrete be deposited preparatory to vibrating?"

"How do you judge when vibration is completed?"

"Do you find it necessary to avoid contact with reinforcing steel and, if so, under what conditions?" (That question is frequent.)

"Have you been able to eliminate air bubbles on surfaces? How?"

"Is it necessary to supplement vibration with spading?"

"Can you suggest a practical test for measuring the effectiveness of equipment?"

"Can segregation be produced by vibration? If so, under what conditions?"

"What favorable or unfavorable effects of vibration have you observed?"

"How does vibration affect the cost of concrete?"

"What changes do you suggest or predict in the near future regarding the application of vibration and the design of vibrators?" Such were the questions to whose answers the committee addressed itself. The papers to be presented answer some of the questions in some respects.

Those papers not published in this issue of the JOURNAL *will appear later and discussions will be welcome until a closing date to be announced at the conclusion of the series*

Practical Application of Vibration

BY C. M. HATHAWAY*

THE USE of mechanical vibrators for placing concrete in the construction of bridges has been required in Illinois for the last year, with very satisfactory results. Internal vibrators are used for this type of construction, giving us a denser and more economical concrete.

An experiment in the use of surface vibrators on pavements was tried in 1932. In brief, it was found that a leaner mix could be used and that a drier consistency was necessary. Also, when the drier consistency was used, the concrete could not be handled very easily by the puddlers and the pavement edges were, at times, honeycombed. Incidentally, surface scaling developed on some of the vibrated sections, but it is uncertain whether this was due to the use of the surface vibrator.

Since our experience has been mainly with internal vibrators, the conclusions given in this paper are based on the use of this type on bridge work. We have found that, in the matter of selecting a vibrator, any machine capable of maintaining 3500 vibrations per minute without straining the power unit, is satisfactory. It is also advisable that both the vibrator and the power unit be of sturdy construction, and easy to handle. Good results have been obtained from both air compressed types of vibrators and those operated by the larger electrical power units. It is preferable to have at least two vibrators on the job, as this will provide a reserve vibrator in case of a breakdown or if the concrete mixing should proceed faster than one vibrator can handle it.

The volume of concrete that can be effectively handled by a vibrator is variable, depending on its size and construction, and also on the consistency and type of materials in the concrete mix. The smaller vibrators can handle from 5 to 6 cu. yd. per hour, whereas the sturdier well-powered vibrators have handled from 15 to 20 cu. yd. per hour. A larger volume of concrete having gravel and fine sand can be handled, within a limited time, than a harsher mix containing crushed rock and coarse sand.

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Our specifications provide for a vibrator that is capable of transmitting vibration to the concrete in frequencies of not less than 3500 impulses per minute, and we have found that such a vibrator gives us excellent results.

Form pressure is noticeably increased when vibrators are used, but the amount of the increase will vary with the amplitude and the frequency. By increasing the number of studs and walers approximately twenty-five per cent, contractors have been able to take care of the additional form pressure. It is absolutely essential that the forms be tight throughout, to prevent the mortar from being forced out through open cracks. Seriously honeycombed surfaces may result if this is not carefully observed. The thickness of sheathing need not be changed if the studs and walers are ample. Most contractors prefer $\frac{5}{8}$-in. or $\frac{7}{8}$-in. ply-board for exposed surfaces, and 1-in. lumber for unexposed surfaces.

Mixes of various combinations have been successfully placed with the use of a vibrator. Those containing gravel and fine sand are handled more easily than those containing harsher combinations of crushed stone and coarse sand, and harsh mixes that could not be satisfactorily spaded by hand have been successfully used when subjected to the treatment of mechanical vibrators.

The slumps of concrete to be vibrated range approximately from $2\frac{1}{2}$ to $3\frac{1}{2}$ in. when small vibrators are used, and from 1 to 2 in. when larger vibrators are employed. The slump on concrete not vibrated is 4 to 6 in. for this type of work.

The vibrators have permitted the sand proportion to be increased up to 15 per cent, due to the fact that the water content could be decreased and still give workable concrete, wihout a resulting loss of strength. Proportions of mixes have been changed so that the yield has been increased as much as 17 per cent.

Vibrators, to obtain the best results, should be applied a few feet back of the deposit. This enables the vibration to be transferred in all directions, thus attaining the maximum effect.

There is no set test for judging when the vibration is completed, except the visual one which is developed by experience. In general, when the concrete mass seems to have "melted" to the point where the mix looks uniform and some mortar has come to the surface, the vibration is considered completed.

It is preferable to have the vibrator kept $1\frac{1}{2}$ to 2 in. from the forms, although it may be placed in contact with the forms if they are well-braced. In the latter case, caution should be used to see that the vibrator does not chip the form.

It is our usual practice to attempt to keep the vibrator from being in contact with the reinforcing steel. In fast placing of concrete, there probably is no harmful effect from such contact, but in slow placing, it is possible that vibration transferred along the reinforcing steel may effect the bond if the concrete about the steel has started to set.

When plyboard is used and the forms are tight, air bubbles develop on the surface of the concrete. These have been eliminated to a great extent by oiling the form lumber and by a supplementary hand spading along the forms after the vibration has been completed. Where bubble holes still exist after the above treatment, they are shallow and will be eliminated easily by rubbing. With this exception, spading is not necessary when vibrators are used, provided the vibrators are of sufficient size and quantity to keep up with the concrete placing.

It is possible to obtain segregation when using vibrators, but not in the ordinary sense in which the term "segregation" is used. In other words, the coarse aggregate will not separate itself from the mortar; but, if the mix has a surplus of mortar, this surplus may be made to come to the top. A reduction of the mortar percentage will remedy this. Vibration increases the bleeding of water, but this water can be removed easily.

With the exception of a possible slight increase in the amount of bubbles on concrete surfaces, we have observed no unfavorable effects of vibration. On the other hand, vibration has enabled us to obtain a denser concrete, giving increased strength, and to eliminate honeycomb.

The use of vibrators cuts down the cost of concrete structures appreciably. There is the initial expense of the vibrators and the slight increase in the cost of building forms, but this is far counterbalanced by the saving of man power in the matter of placement and the saving of cement. Mixes that could not be handled by hand have been easily worked by vibrators. The actual savings depend to a great extent on the contractor and the types of materials, but the percentages can be given roughly as follows: A reduction of from 40 to 70 per cent in the man power required for spading concrete; a decrease of 5 to 10 per cent in the amount of cement required; an increase of 5 to 10 per cent in the cost of form-building. Patching and finishing costs are also somewhat reduced, although not to any great extent, as our mixes in the past have been designed to insure workability and to give us a good product, assuming that ordinary precautions were taken.

As for the future prediction regarding the use of vibrators, the opportunity for improvement always exists in any product. A less

unweildy machine would be of advantage in internal vibrator types. Power units can be made more trouble-proof. The cost of these machines will probably decrease as their use increases.

In conclusion, I wish to repeat that the preceding comments apply almost wholly to the use of internal type vibrators.

Vibrated Concrete in Pavement Slabs

BY F. V. REAGEL*
MEMBER AMERICAN CONCRETE INSTITUTE

Occasionally some development in an art or a science comes along, the inherent merit of which is so logical and so readily apparent that it needs little sales effort and the desire for its use tends to outstrip the development of equipment, methods or facilities for its use. Such seems now to be the position in which we find the vibratory method of placement and finishing of concrete in pavement slabs. Numerous agencies are ready and anxious to take advantage of the benefits but find that the maximum benefits are not yet available nor the most convenient use possible because the development of equipment is yet a step or two behind.

There seems to be little room for doubt in the minds of those who have been using this method experimentally that it is one of the most important developments of recent years in its field. Each succeeding development project confirms conclusions that through the use of this method, either concrete of equal strength can be produced at a lower unit cost, or concrete of greater strength can be produced for the same expenditure, both apparently accompanied by improved general density characteristics which give promise of greater durability. It will be difficult indeed to disregard the evident dividend in sight and contrary to past experience if engineering ingenuity fails to provide the means for realization.

The underlying principle of vibration is perhaps not entirely new. Investigators, a number of years ago, in connection with water cement ratio studies, discovered and stated in effect that concrete of the maximum strength and density was not obtainable within the range of plasticity and with a gradation of coarse aggregates compatible with maximum workability but that these maxima were to be found somewhat outside this range on the side of the more coarsely graded aggregate. Present investigators are finding, and it is now, in the opinion of the author, proven, that the water cement ratio law holds true for vibrated as well as for unvibrated concrete provided the vibrated mix *becomes* plastic.

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Vibration, therefore, is simply a tool, the use of which is to permit the handling, deposition, and finishing of concrete mixes otherwise too harsh and unworkable for use. As such a tool, however, its development is far from perfect and certain limitations or deficiencies become apparent quickly with use. We shall briefly list and discuss a few of these limitations which have come to our attention in connection with the development projects thus far carried on under our observation.

Two general types of vibrating equipment have been used thus far in connection with our work. One type operates as a surface vibratory screed traveling on and operating from the pavement forms and the other is of the so-called pan vibrator type which tends to vibrate more deeply into the mass of concrete without any direct transmission of vibration to the pavement forms. The use of each type, in its present stage of development, is attended with certain limitations and difficulties.

Both types of equipment are sharply limited as to the range of consistency of concrete which can be satisfactorily handled. The probable optimum range of consistency at present is from ¾-in. to 1½-in. slump at time of finishing. This range may vary with special mix conditions being largely dependent upon the ratio of the mortar content to the void content of the coarse aggregate. If a mix containing too great a percentage of mortar or of too high a slump is used, the excess mortar pumped to the surface will be objectionable and in some cases will even stop the mechanical operation of the machine because of the accumulated volume. Also a serious waste of mortar and loss of time in finishing operations will result. If the opposite end of the range is passed, objectionable segregation will occur during distribution of the concrete and the desired density will not be obtained. If too large a quantity of this type of mix is placed, the screed cannot cut off the excess and finishing difficulties again arise.

Another specific difficulty arising from the characteristic mix used under this method, is the difficulty of finishing the lip placed at the edge of the slab with lip curb design. This is a rather troublesome matter to the contractor and an auxiliary vibrating finishing mule or shoe or the use of a special plastic mix for use in placing the lip is recommended.

For the general control of the consistency characteristic of the mix, the Bureau of Public Roads has suggested as a safe rule the use of the driest concrete which will become plastic when vibrated, the plasticity under vibration being the critical measure of workability rather than the plasticity of the mixed concrete.

Directly related to the foregoing is the range of allowable moisture content in aggregates entering into the mix. Since the drier range of consistency must be used to obtain the benefits of vibration, sudden and extreme variations in aggregate moisture content must be avoided if any reasonable control is to be obtained. The use of stockpiled material, and, if possible, the use of multiple sized aggregate is recommended for ease of control in this connection.

Even after optimum control of consistency is obtained, there is inherent in the type of mix used, a tendency toward segregation and consequent honeycombing of the slab. The mix is deposited at present by gravity without agitation and the coarsest particles of the aggregate composing the harsh dry mix naturally tend to roll to the side of the batch as deposited. This material is met by the same action from the adjacent batches. This tendency persists into the finished slab to some degree at least as is shown by the consistent difference between beams taken from various locations across the pavement slab. From one experiment performed by this Department a difference in strength of approximately 14 per cent was developed from beam averages, the beams being taken parallel to the centerline of the pavement and at intervals across the slab. The lowest strength concrete was obtained next to the outside form and the highest at about the quarter point of the slab. Cores taken methodically from the slab also confirm the tendency toward a difference in strnegth and texture apparently related to the persistence of the effect of the original hills and valleys formed by the original batch deposition. There is no equipment at present developed for the vibration of pavement slabs, in so far as we know, that has any promise of drawing or distributing mortar horizontally through a slab of concrete although they all apparently function excellently in a vertical direction. Consequently there is need for some means of depositing this type of mix in a wider ribbon and preferably in an agitaged condition until deposited.

The type of mix which is advantageously handled by vibrator equipment is, due to lack of plasticity, very difficult to strike off and level for the placement of reinforcement and requires considerably more man power for placement. The development of some means of mechanical strike off is desirable in this connection unless some other solution presents itself.

In this connection, an interesting speculation on design presents itself. Since this type of mix presents difficulties in the placement of a layer over reinforcement and at the same time shows considerably decreased shrinkage characteristics, perhaps further development

will indicate the possibility or expedience of substituting adequate crack control for present reinforcement and either utilizing the difference in cost in increased thickness or as a saving.

Returning to specific limitations of the two types of vibratory equipment so far used by this Department, we find, in the case of the vibratory screed, that the main objectionable feature is the fact that the direct transmission of vibration to the forms introduces some difficulty in maintaining an accurate form level, indicating the need for stabilizing form bearings where needed or the bridging of lack of bearing by more rigid and stronger forms. The "bull nosed" vibrating screed can, however, apparently compact and finish harsher mixtures of concrete having slightly drier consistency than is possible in the case of the pan type vibrator as now designed and assembled. In addition it is possible to carry some head of material in front of the screed thus automatically providing for the desired thickness if the forms hold true, whereas the use of the pan type vibrator, as now assembled, requires the correct estimation of the quantity of concrete to maintain the proper cross-section of slab. If incorrectly estimated or applied in non-uniform cross-section, correction is very difficult after the mass has been vibrated. A possible improvement in the pan type might result from the use of a heavier pan placed in the front of the finisher in such a manner as to permit some strike-off action. In any event an attempt to carry too much excess material ahead will result in choking down the finisher. A more promising solution to the whole problem may be found in the field of internal vibration if this method can be adapted conveniently for use with pavement slabs.

In conclusion we desire to state that the above remarks are not intended as condemnation of the equipment now available or even as a comparison of the two particular types of equipment used. Rather, our effort has been to point out some of the present difficulties to demonstrate the need of improved design if volume use of this method is to be obtained. Each type of equipment has produced good economical concrete pavement on our projects.

No type of equipment can, in our opinion, be expected to remedy all defects of mixes or even of placement, after the mix has been deposited on the subgrade, unless perhaps internal vibratory equipment can be developed. The only action of the surface vibratory types, in so far as the lower portion of the slab thickness is concerned, is to compact the mix and force the excess air and liquid portion to the surface or in other words, to obtain maximum density as placed. The method

does, however, permit and even necessitates the use of concrete mixtures which are commonly considered too harsh to be workable under the conditions of ordinary present day pavement construction methods. These mixes contain a greater percentage of the cheaper ingredient and a smaller percentage of the most expensive ingredient and thus open up new possibilities of design economy. It is predicted that further use and study of the vibratory method will pay increasing dividends to the user.

Vibration of Concrete

BY W. R. JOHNSON*
MEMBER AMERICAN CONCRETE INSTITUTE

In answering the questionnaire submitted by the committee on vibration, the writer has attempted to state as briefly as possible the answers to some of these questions based on his experience with vibration, both in field and laboratory during the last six years.

The selection of a vibrator for a particular job is dependent mainly on the type of form in which the concrete is being placed.

In large open forms an internal or platform vibrator is recommended. Both of these types are being used successfully in placing of this character, although the present trend is more toward the internal rather than the platform vibrator. In placing in large open forms the size of the batches generally ranges from 2 to 6 cu. yd., with aggregate running from 1½- to 9-in. maximum. A two-man vibrator is best suited for this type of work.

Internal vibrators also should be used in walls and reinforced sections. In general it might be said that the internal vibrator can be used in any type of construction where it is possible to insert the vibrator into the concrete.

Form vibrators should be used where it is impossible to use either the internal or surface type, such as heavily reinforced thin walls, tunnel linings or concrete pipe. In using a vibrator of this type forms must be tight and well built as this machine is hard on form work and also has a tendency to pull the mortar out of the concrete through any small openings.

The volume of concrete that can be effectively handled per vibrator varies with the size and type of vibrator. In general the quantity varies from about 12 to 36 cu. yd. per hour.

Form work need not be changed to any great extent in the placing of mass concrete as the internal or platform vibrators used do not come in contact with the forms. As stated before, however, all forms should be made as tight as possible to prevent leakage. This is

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particularly true in heavily reinforced thin sections where the concrete is necessarily wetter than in large open forms.

The consistency of concrete placed by vibration should be considerably dryer than concrete placed by ordinary methods, such as booting or spading. For example, concrete placed in large open forms and booted in place requires a slump of 2 to 3 in. whereas, placing with vibration in a similar form requires a slump of only ½ to 1 in. Similarly, concrete requiring 6- to 8-in. slumps for placing by ordinary methods can be placed with a 2- to 4-in. slump by vibration.

For vibrated concrete the sand proportions can be somewhat reduced, particularly for mass work. For example, a mix using 35 to 37 per cent sand by weight with ordinary methods of placing can be safely reduced to 31 to 33 per cent when the concrete is vibrated. This reduction can best be accomplished by holding the mortar proportions constant and increasing the coarse aggregate. In other words, a mix of 1:2.4:4.4 by weight can be changed to a 1:2.4:4.9 by weight if the concrete is vibrated.

In vibrating mass concrete the vibration should be applied at the point of deposit and should be worked until the mass is flattened and the coarse aggregate is thoroughly incorporated into the mortar. Determining the end point of vibration is a matter of experience which is soon acquired by the foreman or inspector.

If reinforcing steel is well tied into place, contact with the vibrator is beneficial rather than detrimental as it helps transmit the vibration into the concrete.

By supplementing vibration with spading most air bubbles can be eliminated from the surfaces of concrete.

The most practical test the writer knows for measuring the effectiveness of equipment, is to work two or more vibrators of different types in the same form at the same time. This will give a direct comparison in effectiveness of placing, but will not give a comparison of up-keep or repair costs unless the work is carried on for a considerable period.

In making a comparison in cost between vibrated and unvibrated concrete there are a number of factors to be considered. Placing costs of vibrated concrete are somewhat higher than for concrete not vibrated. The additional costs include extra labor, investment in and up-keep of vibrators. Form costs are practically the same. Patching and finishing costs are practically the same. Material costs, that is, cement, are lower with an average saving of one-half to one bag of cement per cubic yard of concrete. The smaller quantity of cement

required, more than compensates for the added placing costs and represents a considerable saving.

With the lower costs of vibrated concrete must also be considered the generally higher quality of the finished product. The dryer concrete used in vibration eliminates to a great extent the excess water, laitance and segregation so common in the past with ordinary methods of placing. This means that the quality of concrete throughout the entire height of a lift will be the same and that there will not be a plane of weakness or a porous concrete at the top of a lift which is subject to disintegration due to weathering. The lower water-cement ratios give greater strengths and durability. The decreased cement content, together with the lower water-cement ratios, also cause smaller volume changes. Generally speaking then, vibration of concrete is better from all view points than placing by the old methods.

Some of the changes that will probably take place in vibration in the near future are a reduction in weight of equipment, higher speeds and possibly greater amplitudes all of which will tend to increase efficiency in placing concrete.

Other papers of this series (see page 417) will be published in the next issue of this JOURNAL

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's corps of Reviewers.

Notes on building regulations

FREDERICK RINGS, *Concrete and Constructional Engineering* (England), Vol. XXX, No. 2, Feb. 1935, p. 120-123. Reviewed by GLENN MURPHY.

Miscellaneous comments on the London Building Act and London County Council Regulations with reference to foundations, strength of core concrete, suggestions for improving regulations, and administration of building acts.

Voids and yield losses in concrete

J. E. BOYD, Testing Engineer, Georgia State Highway Dept., *Engineering News-Record*, Vol. 114, No. 5, Jan. 31, 1935, p. 156. Reviewed by N. NEWMARK.

This paper describes tests on cured concrete to determine the actual voids. Comparison is made between the voids determined experimentally and the theoretical voids obtained from the water used and the amount of water required for hydration of the cement.

Logarithmic charts for reinforced concrete sections

J. P. PORTER, *Concrete and Constructional Engineering* (England), Vol. XXX, No. 2, Feb. 1935, p. 93-103. Reviewed by GLENN MURPHY.

Instructions for constructing and using logarithmic charts for design of rectangular reinforced concrete beams with or without compression steel, and for T-beams without compression steel. An illustrative chart accompanied by examples is given for $f_s = 18{,}000$ p. s. i., $f_c = 900$ p. s. i. and $n = 15$.

The viaduct of Esla

CESAR VILLALBA GRANDA, *Hormigon Y Acero* (Madrid), Vol. 1, No. 8, Dec. 1934, p. 371-376. Reviewed by MILES N. CLAIR.

The second of a series on the design and construction of a concrete arch railroad bridge of approximately 650 ft. span now under construction. This article deals with the design of the wooden false work to support the forms. Sketches of the false work and construction photographs are given.

Line load action on thin cylindrical shells

HERMAN SCHORER, *Proceedings, Am. Soc. C. E.*, Mar. 1935, Vol. 61, No. 3, p. 281 Reviewed by H. J. GILKEY

The thin cylindrical shell is due for increased use in engineering structures because of its obvious economy in many instances. The paper is a mathematical treatise outlining a method of analysis of an elastic shell. The method is based upon simplifying assumptions which result in a useful approximation but one that has definite limitations which need to be kept in mind.

The new code applied to design

C. E. Reynolds, *Concrete and Constructional Engineering* (England), Vol. XXX, No. 2, Feb. 1935, p. 136-149. Reviewed by Glenn Murphy.

Continuation of a series. This article is under the heading "Upper Floors with Slabs Spanning in Two Directions," and deals with floor slabs supported on four sides with or without provision for holding down the corners, and reinforcement to resist torsion in the corners. Moment reduction coefficients are discussed, and examples and tables are included.

Argentina roads

Report—National Bureau of Roads of the Republic of Argentina, Buenos Aires, 1933. Reviewed by Miles N. Clair.

First report of the Bureau which was created in 1932 comprises two volumes. Volume 1 of over 600 pages gives the history, organization and data on the road system at present and plans for the future. There are included maps, charts, photographs and tables giving data on traffic counts, types of roads, bridges and costs. Volume 2 of over 250 pages gives the laws relating to the use and construction of highways.

Bituminized concrete casing for piles in sea water

Engineering News-Record, Vol. 114, No. 4, Jan. 24, 1935, p. 121. Reviewed by N. Newmark.

In the construction of the Bellamy River Bridge in New Hampshire special attention was given to the concrete pile trestle 660 ft. long to prevent disintegration. Impregnating whole piles with bitumen calls for a plant beyond the means of small operations, and bituminized concrete casing slabs were developed to simplify the protection procedure and reduce cost by limiting the protection to the parts of the pile needing it. The article describes also the construction and driving of the concrete piles.

A system of low level streets

Pascual Palazzo, Pamphlet, Buenos Aires, 1933. Reviewed by Miles N. Clair.

The author, a prominent road engineer, in a pamphlet of approximately 90 pages, offers a plan to overcome the problem of urban traffic. He discusses the tremendous loss of life and property due to accidents and the economic loss due to traffic delays. Basic principles are cited and a solution of the traffic problem is given as the construction of wide multi-lane streets at different levels so that they do not cross at the same level. The plan contemplates also delivery to the stores, etc., by low level streets. The cost of the plan and its financing are worked out in considerable detail.

Tests of reinforced concrete columns

Otto Graf, Bulletin 77, German Reinforced Concrete Committee, 1934. Reviewed by Inge Lyse.

This report contains the results of an investigation of square and round tied and spirally reinforced concrete columns. Both direct loading to failure and sustained loading were used. The results showed perfect agreement with the findings of the American Concrete Institute's investigation. The only additional information is the variation found in the ratio between strength of concrete in the columns and strength of control cubes. This ratio varied from as high as 0.75 for concrete of strength of 2100 p. s. i. to as low as 0.50 for 8500 p. s. i. Tables and diagrams give all test data.

Stabilizing constructed masonry dams by means of cement injections

D. W. COLE, *Proceedings, Am. Soc. C. E.*, Feb. 1935, Vol. 61, No. 2, p. 187.
Reviewed by H. J. GILKEY

Describes the methods and results obtained by injecting 64,000 bbl. of portland cement, as grout, into 380,000 lineal feet of drill holes in three rubble masonry gravity dams located in India. Dams were from 30 to 190 ft. in height and had been in service since 1917. Serious leakage had developed. Grouting was performed under full reservoir conditions. There is some discussion of the stability of gravity dams in general.

Reinforced concrete chimney and gas washing tower at Deptford

Concrete and Constructional Engineering (England), Vol. XXX, No. 2, Feb. 1935, p. 104-106.
Reviewed by GLENN MURPHY.

Short description of the construction of the structure recently erected at Deptford West Power Station. The chimney is supported on the gas-washing tower, a brick-panelled steel structure with beams and stanchions encased in aluminous cement concrete. The foundations are 30 ft. below ground level and the chimney has a height of 285 ft. The internal diameter varies from 20 ft. 9 in. to 17 ft. 9 in. and the thickness of the reinforced concrete varies from 6 in. to 10 in.

Comparison of portland cement specifications of the world

M. WATSON and Q. L. CRADDOCK, *Cement and Cement Manufacture*, Vol. 8, No. 1, Jan. 1935, p. 16-21. Reviewed by J. C. PEARSON.

This is the first of a series of tables to be published in this journal comparing the requirements of the standard specifications for portland cement issued in 28 countries. Four tables are presented in this issue: Table I, on Fineness, gives the date of the specification, data on sieve openings and wire diameters, and permissible maximum residues for both rapid hardening and ordinary cements; Table II gives chemical limitations, hydraulic moduli and specific gravity; Table III gives times of setting (Vicat method) and tests for soundness; Table IV gives tensile and compressive test requirements (in lb. per sq. in.).

Planning bridge-pier work for efficient concreting

Engineering News-Record, Vol. 114, No. 9, Feb. 28, 1935, p. 305. Reviewed by N. NEWMARK.

A plan involving several batching plants with small tip-over-body trucks delivering dry materials to mixers at the various pier sites was adopted for concreting the substructure for the two highway bridges being built across the Cape Cod Canal in Massachusetts. The job required 70,000 cu. yd. of concrete involving a dozen land piers and abutments some of which extend 100 ft. above the ground surface, on two bridges located 4 miles apart. It was necessary to provide for 4300 cu. yd. continuous seal pours for each of the four piers at the water's edge at an average rate of 133 cu. yd. per hour. A description of the layout of the batching plants and the construction procedure used is given in the article.

Tests on reinforced brick masonry columns

M. O. WITHEY, *Proceedings, Am. Soc. for Testing Materials*, Vol. 34, Part II, p. 387.
AUTHOR'S SYNOPSIS.

Tests of thirty-two 12½ by 12½-in. brick columns 6 ft. high made with Chicago common and with Waupaca face brick are reported. The variables included three percentages of longitudinal and a like variation in lateral reinforcement together

with two high-strength portland cement lime mortars. The results indicate that the column strength varies directly with the strength of the masonry and the percentage of longitudinal steel, and is increased by the use of lateral reinforcement. Strengths of 5000 p. s. i. based on gross area and over 7000 p. s. i. on the cores after shell spalling were attained. For suitable reinforced brick masonry columns a factor of safety of 3.5 to 4.0 is suggested.

The vane calorimeter

R. W. CARLSON, *Proceedings, Am. Soc. for Testing Materials*, Vol. 34, Part II, p. 322.
AUTHOR'S SYNOPSIS.

This paper describes a type of calorimeter which can be used for measuring heat of hydration of neat-cement pastes while curing at substantially constant temperature. This calorimeter has certain advantages over other types of calorimeters, chief among which are its low cost and its suitability for cements which are not readily soluble in acids.

Typical data are presented with diagrams showing the hourly rates of heat evolution for a number of cements plotted against the time after mixing. Comparison of results from this calorimeter and from the heat-of-solution calorimeter indicate that satisfactory accuracy is obtained using the vane calorimeter.

Concrete experimental work under desert conditions

Engineering News-Record, Vol. 114, No. 4, Jan. 24, 1935, p. 118. Reviewed by N. NEWMARK.

The article describes tests being made by the Metropolitan Water District of Southern California to determine the kind of concrete to withstand the severe conditions along the Colorado River Aqueduct. Studies are being made of 300 small concrete slabs exposed to the weather in a typical desert location and of 200 similar slabs subjected to various extreme conditions of temperature and wetting and drying in the laboratory in an attempt to determine the effects of exposure in the desert at a more rapid rate. Methods of curing, resistance to sulphate action, aggregate tests, and effect of color of concrete are other topics given major consideration in the experimental program.

Vibration and previbration in concrete depositing

T. J. GUERITTE, *The Structural Engineer* (England), Vol. XIII (New Series), No. 2, Feb. 1935, p. 92-105. Reviewed by V. P. JENSEN

Historical developments and references are given. Vibrators are described and a few tests are summarized. Generally applicable quantitative conclusions are avoided but qualitative observations taken from Mr. Deniau are commended. Thus,

1. A few seconds of vibration secure a very marked subsidence of the concrete, which may reach about one-fifth of the depth of concrete. If vibration is then stopped, it will be found when removing the shutters that a very good face has been obtained.

2. Further vibration brings about a kind of liquefaction of the concrete and its de-aeration, many air bubbles bursting when reaching the upper face of the concrete.

3. Further vibration may bring about segregation.

Masonry and reinforced concrete

HALE SUTHERLAND, Chairman, Progress Report of Committee of the Structural Division, Am. Soc. C. E., *Proceedings, Am. Soc. C. E.*, Mar. 1935, Vol. 61, No. 3, p. 341. Reviewed by H. J. GILKEY

Consists of brief reports from each of the following sub-committees:

A. Design of Rigid Corners in Reinforced Concrete.

B. Reinforced Concrete Frames under Wind-Load.

C. Volume Changes and Plastic Flow in Concrete. (An excellent outline of the

factors involved).

D. Reinforced Concrete Columns.

E. Reinforced Concrete Slabs supported on Four Sides.

F. Moment Coefficients for Irregular Types of Flat Slabs.

Several of the reports are merely statements that work is in progress.

Design of piled jetties and piers

OVE ARUP, *Concrete and Constructional Engineering* (England), Vol. XXX, No. 2, Feb. 1935, p. 107-113. Reviewed by GLENN MURPHY.

Last of a series of articles in which the author discusses the design of piers. This number gives an example of how the theory has been applied on a jetty built for the United Glass Bottle Manufacturers (Charlton), Ltd., at their Charlton Works. The jetty is L-shape in plan with a shore arm extending 170 ft. into the river, and a river arm 160 ft. long. It has a concrete deck 28 ft. wide carrying two standard-gage tracks with a turntable at the corner of the L, and is designed to carry two 40-ton cranes with a 5-ton load on an 18 ft. jib, 25-ton railway cars, and 12-ton trucks. The deck is about 33 ft. above the river, and is supported on 16 in. by 14 in. by 50 ft. reinforced concrete piles.

Covering over the River Mersey

Concrete and Constructional Engineering (England), Vol. XXX, No. 2, Feb. 1935, p. 114-119. Reviewed by GLENN MURPHY.

Design features and progress report on construction details of a project for ultimately covering over about 1400 ft. of the river to provide a new east-west street which will relieve some traffic congestion. The project will also involve replacing the old Mersey bridge with a new reinforced-concrete portal frame. At present there are seven open-spandrel arch ribs 22 ft. 3 in. o. c. with spans of 87-90 ft. The ribs have a rise of 26 ft. 6 in., a width of 3 ft. 3 in. and a depth varying from 3 ft. 3 in. at the springing to 2 ft. 6 in. at the crown. The crown reinforcement consists of twelve 1⅝ bars top and bottom. The footings are 12 ft. square and rest on sandstone.

Brinell test applied to cementing materials as a test of strength

QUIRINO SESTINI, *Le Strade* (Milano), Vol. 16, No. 7, July 1934, p. 381-389. Reviewed by MILES N. CLAIR.

An investigation to determine the relation between the compressive strength of cement paste or "normal" mortar and the Brinell hardness test. Comparisons were made for 22 cements of various types. Nine diagrams are given showing the relation of the diameter of the impression made by the Brinell test and the compressive strength of the standard mortar. It is concluded (1) that the diameter of the impression made by the Brinell test was not a constant function of the compressive strength for strengths below 7900 p. s. i.—(2) that for strengths above 7000 p. s. i. there was no relation between the diameter of the Brinell indentation and the compressive strengths—(3) that it was impossible to use the diameter of the Brinell impression as a measure of the compressive strength of a cement paste or mortar. The charts show a straight line relation with diameter of indentation decreasing with compressive strength of mortar down to some value near a mortar strength of 7000 p. s. i. where the diameter remains constant irrespective of the compressive strength.

Engineering problems of the future highway

Engineering News-Record, Vol. 114, No. 3, Jan. 17, 1935, p. 69 to 106. Reviewed by N. NEWMARK.

The annual highway number of the *News-Record* contains a symposium of eight articles concerning highway problems and development. R. W. Crum, Director of

Highway Research Board, National Research Council, gives a review of highway research. A description and discussion of traffic surveys is presented by Miller McClintock, Director of Bureau of Street Traffic Research, Harvard University. A discussion of the rerouting and rectifying of existing routes is given by the State Highway Engineer of California, C. H. Purcell. Rational road design is outlined very fully by Frank T. Sheets, Consulting Engineer of the Portland Cement Association. A survey of the important phases of highway bridge design is given by J. R. Burkey, Chief Engineer of Bridges of the Ohio Highway Department. M. W. Torkelson, Director of Regional Planning of the Wisconsin Highway Commission discusses roadside design. B. E. Gray, Chief Highway Engineer of the Asphalt Institute writes of secondary roads, and E. W. James of the U. S. Bureau of Public Roads is concerned with highway safety.

A suspension turbidimeter for determination of specific surface of granular materials

ALEXANDER KLEIN, *Proceedings, Am. Soc. for Testing Materials*, Vol. 34, Part II, p. 303.

AUTHOR'S SYNOPSIS.

This paper describes a portable apparatus for the rapid determination of the surface area of portland cement and other granular materials, in which determination the turbidity of a suspension of the material in castor oil is measured. The method does not involve Stokes' law which is the basis of sedimentation methods, but consists in determining the mean effective cross-sectional area of the suspended material under static conditions, through the use of photoelectric equipment. Hence, no consideration of time or velocity is involved in the determinations, no standard samples are required, and the method is independent of the physical characteristics of the suspending medium. A theoretical constant required for the analysis of a given material is determined from an observation on a sieved sample of the material itself. A high degree of reproducibility is attained and the method appears to furnish an accurate measure of the relative fineness of various materials.

Comparisons are made between test results obtained by means of this apparatus and by means of apparatus employing the principle of sedimentation.

Study of cement composition in relation to strength, length changes, resistance to sulfate waters and to freezing and thawing of mortars and concrete

H. F. GONNERMAN, *Proceedings, Am. Soc. for Testing Materials*, Vol. 34, Part II, p. 244.

AUTHOR'S SYNOPSIS.

This paper presents data obtained over a period of three years in an extensive cooperative investigation between the Research Laboratory of the Portland Cement Association, Chicago, and the Portland Cement Association Fellowship at the National Bureau of Standards, Washington.

In this investigation 121 laboratory-prepared portland cements, covering 72 different compositions and ground to essentially the same fineness, were studied in relation to strength, length changes, resistance to sulfate waters and to freezing and thawing, in mortars and concrete. In this paper the results of tests on 74 of the cements are presented which include several compositions covering the effect of variations in MgO content.

The results reveal that mortar and concrete properties are markedly influenced by the compound composition of the cement. The effects of each of the four major cement compounds on the various properties investigated are discussed. An extensive list of references is added.

Fatigue and flat slab vibration tests

"Repeated compression stresses in concrete," O. GRAF and E. BRENNER and "Vibration tests of reinforced concrete flat slabs," W. GEHLER and W. HORT, Bulletin 76, German Reinforced Concrete Committee, 1934.
Reviewed by INGE LYSE

The first one of these papers reports the results of fatigue tests in compression of concrete specimens. One group had specimens of about 5 in. sq. cross section by 16 in. long, while another group had 6-in. diameter cylinders of 20 in. length. The two types of specimens gave substantially the same results. The fatigue limit was reached at about two million repetitions of loading. For a variation in stress between zero and the full value, the fatigue limit varied from 61 to 66 per cent of the compressive strength. For variation in stress between 75 per cent of the full stress and the full stress, the fatigue limit varied between 77 and 87 per cent, and when the lower stress limit was about 95 per cent of the upper, the fatigue limit reached 92 per cent of the strength. The results indicated that when the repetition of stress took place at a low rate (10 per minute) the fatigue limit was lower than at a high rate (450 per minute). The fatigue limit (in per cent) seemed to decrease with the increase in strength of the concrete. Cracks would appear in the specimens before failure and certain specimens developed cracks without failing.

The second paper describes vibration tests of a flat slab of the Ford Motor Co.'s new factory in Köln and the results are presented in a number of diagrams.

Tests on machine made reinforced concrete members

W. GEHLER and H. AMOS, Bulletin 75, German Reinforced Concrete Committee, 1934.
Reviewed by INGE LYSE

The principal purpose of the reported investigation was to study the load formula given in the German building regulations for reinforced concrete structures. The formula is as follows: $P_W = \frac{B-2G}{3}$ where P_W = permitted maximum load, B is the load at failure, and G is the dead weight. A number of beams of various cross-sectional shapes and a number of different types of plate sections were tested. The results showed that the first crack appeared when the load varied between 0.9 and 1.0 of the permitted load for beams and between 0.8 and 1.4 for plates. The factor of safety based on ultimate load varied between 2.1 and 2.5 for beams and between 3.1 and 3.6 for plates. The observed deflections at permitted maximum moment were greater than the computed values, varying between $\frac{1}{950}$ and $\frac{1}{1550}$ for beams and between $\frac{1}{380}$ and $\frac{1}{2400}$ for plates. The observed stresses at maximum permitted moment were less than the permitted values and varied greatly between beams of different shapes. The tensile stresses in the concrete caused a marked reduction in the stress in the reinforcement. The report contains 45 tables and 27 figures.

Proposed method of making compression tests on portions of concrete beams from flexure tests

L. H. KOENITZER, *Proceedings, Am. Soc. for Testing Materials*, Vol. 34, Part II, p. 406.
AUTHOR'S SYNOPSIS.

The practice of making flexure tests upon concrete for use in pavements is now almost universally established. In addition to the modulus of rupture strength, most engineers, however, also desire to secure the compressive strength of the concrete. The Kansas Highway Commission has for several years been determining the compressive strength of the concrete on paving jobs by sending in to the main

laboratory portions of the specimens broken in flexure, the ends of which are cut square by a silicon carbide saw. The prisms are then capped and tested in compression. Some states have followed a similar procedure, while other states have continued to make the standard 6 by 12-in. cylinders along with the flexure specimens. Others have drilled out cores or ground down beam ends and made compression tests.

About four years ago, it came to the attention of Prof. C. H. Scholer that in some European laboratories it was common practice to make compression tests upon portions of beams from flexure tests (as modified cubes) as described in this paper. The Road Materials Laboratory of the Engineering Experiment Station of Kansas State College, in cooperation with the Kansas Highway Commission, has been making a rather informal investigation of this method and the results are herein presented.

The compressive strength of the modified cubes is closer to the compressive strength of the standard 6 by 12-in. cylinders than is the compressive strength of prisms.

Progress in reinforced concrete in 1934

Various authors, *Le Constructeur de Ciment Arme*, Nos. 184 and 185, Jan. and Feb., 1935.
Reviewed by P. H. Bates

All of the January number of this journal and much of that of February are devoted to contributions by various authors on reinforced concrete. Cements, are considered by Feret and Dumesnil, the former briefly discussing cements in general, whereas the latter is concerned with metallurgical or slag cements. On steel, Professor Saliger, confines most of his article to the use of high tension steels.

In the field of outstanding works we have contributions from Maison Hennebique. Of interest here is also a brief resume of the history of this firm so much concerned with the introduction of reinforced concrete. The firm of Limousin describes a very large concrete wind tunnel erected at one of the aerodromes in France. The firm of Franki briefly summarized some of the pile work they have done in many parts of Europe. Professor Magnel briefly describes investigations in Belgium on thermal contraction of concrete, the influence of sands on the quality of concrete, and a study of vibrated concrete. Doctor Probst contributes an article on the actual state of our knowledge of reinforced concrete. Marcotte of the Testing Laboratory at Ponts et Chaussees departs somewhat from the general trend of the articles to indicate what should be done in future investigations as a result of the progress during the last year.

Three other articles cover in more detail the question of vibration of concrete. Pumping of concrete is covered by Londais. The calculations incident to concrete design are described briefly in an article by Lievin, which covers the subject of monolithic construction. The editor indicates that this symposium has not been completed but later numbers of this journal will add other contributions.

A novel bridge in Hamburg

Horn and Peschges, *Beton und Eisen*, Vol. 34, No. 1, Jan. 5, 1935, p. 6-9.
Reviewed by A. A. Brielmaier

The bridge replacing an old timber structure over the South Canal in Hamburg has several unique features. It is in a congested area where very little space is available for construction purposes. In addition, adjacent buildings without suitable foundations on filled ground, show extensive cracking. The usual type of crossing with steel girders on heavy end abutments and an intermediate pier, all on wooden piles, was therefore not feasible.

The adopted design consists of concrete girders with a total length of 90 ft., continuous over two spans. The center support is on wooden piles. At the ends of the

bridge 12-in. steel H sections were steam driven to a depth of 20 ft. below the canal bed. The cellular openings between the piles were concreted.

At the three supports the girders have vertical extensions similar to enlarged haunches, to a depth of 6.5 ft. below the girder soffit, where a hinged connection is formed with the supports. The hinge is effected by the use of reinforcement crossing at the center of the bearing area. Any earth pressure not resisted by the pile bulkheads is transferred to the center pier.

To determine whether there would be any undesirable movements under loads, due to the comparatively flexible abutments, a loading test was made with nine heavy water trucks of the street department. No movements were observed in the abutments.

The appearance of the bridge is pleasing and not unlike the rigid frame structure known in this country.

Properties and manufacture of sand lime bricks

ANON. *Concrete Building and Concrete Products*, Vol. X, No. 1, Jan. 1935, p. 7-12.

Reviewed by J. C. PEARSON.

The interest in this paper for concrete products manufacturers lies chiefly in the description of the high pressure steam curing equipment and plant layout of an up-to-date sand lime brick plant in England—Sevenoaks Brick Works, Ltd., at Greatness, Sevenoaks, Kent. This plant has a capacity of 20 million bricks per year, and is equipped with four autoclaves capable of accommodating 62000 bricks at a time. The bricks are steamed for 15 hours at a pressure of 140 p. s. i., after which treatment they are ready for use. Transverse tests of the finished product are frequently made to insure compliance with the British Standard Specifications. A typical figure for modulus of rupture is 750 lb. tested dry; slightly lower values are obtained on saturated bricks.

Sand is hauled in by trucks, and dumped through a grating into an underground bin, whence it is carried by elevator to a hopper discharging on the ground floor. Lump lime is ground to pass a 40-mesh sieve in a ball mill, and conveyed to a hopper beside the sand hopper. These hoppers discharge to conveyor belts running toward each other, the lime belt being higher than the sand belt, and discharging on top of the latter. The proportion of lime to sand is between 8 to 92 and 10 to 90. The mixture is then elevated to the top of the building and passes into a rotary steaming drum, where in the course of 20 minutes or so the lime is hydrated. Thence the mixture goes to a rod mill, from which it is conveyed to revolving presses. Here the dampened mixture is pressed in brick molds with pressures of 100 to 200 tons per brick, after which the bricks are stacked on special trucks and wheeled to the autoclaves.

A new type of form for circular conduits

E. MARQUARDT, *Beton und Eisen*, Vol. 34, No. 1, Jan. 5, 1935, p. 9-12.

Reviewed by A. A. BRIELMAIER.

The Cravetto (patented) method of forming continuous concrete conduits with circular sections up to 3 ft diameter, has been in use in England and on the Continent for four years. The inside form consists of a tube of flexible, impervious material such as rubber or a cloth, with an outside diameter equal to the opening of the conduit. This tube is placed within the outside forms and expanded to the desired size by water or air pressure at about one-third to one-quarter atmospheric pressure. The

pouring proceeds in lengths of about 300 to 400 ft. At the end of each section a length of steel tubing is placed partly within the section poured and extending into the adjacent section which will be poured later. This tends to prevent damage to the fresh concrete at the junction of pours. The arrangement for removing the pressure tube is of particular interest. A rope or steel cable attached to the end of the tube, extends through the center to the other tube end. When the form is to be removed, the pressure is slightly reduced and the tube is removed by pulling this attached line. In this operation the tube is turned inside out, avoiding any danger to the concrete through sudden or interrupted movements.

A crew of 8 men can pour about 300 ft. of 16-in. conduit in an 8-hour day using this method of forming. The tubes are left in place under pressure overnight and removed and reused the following day. The wear on the tube is slight. In Marseilles, where the method was first applied, a tube was used daily for almost two years.

In connection with this system of forming, the interior of the conduit may be coated with a surface protecting material by pressure spraying. About 3000 ft. of conduit can be coated in one work day by a 5 man crew.

Tests of elasticity, plasticity and shrinkage of concrete

W. Gehler and H. Amos, Bulletin 78, German Reinforced Concrete Committee, 1934.

Reviewed by Inge Lyse.

Stimulated by Freysinet's experiments with concrete specimens subjected to long time loading and by the American Concrete Institute's column investigation the German Reinforced Concrete Committee in 1931 initiated an investigation of the elastic and plastic properties of concrete, including shrinkage observations. The report presented in No. 78 covers the first year of test. Reinforced concrete beams subjected to eccentric longitudinal loading which produced a constant moment over the full length, were used as test specimens. The results of these tests, which were considered as pilot tests for a larger investigation, are presented in tables and diagrams and may be summarized as follows:

1. The method of test used was found to give consistent results, indicating superiority over column tests.

2. The plastic deformations of the concrete became as much as eight times as great as the initial load deformation, while the strain in the reinforcement remained practically constant.

3. The plastic deformations are not caused by shrinkage alone, as the total deformations were more than twice as large as shrinkage deformations.

4. The plastic deformation had practically ceased at the end of five months. At one year the deformation became fully stationary.

5. The recovery by unloading corresponded very well with the initial load deformation, the plastic deformation remaining as permanent set.

6. Reloading produced only elastic deformations.

7. The flexural strength at one year was not affected by previous plastic flow.

8. Since a high-early-strength cement was used, the specimens were subjected to loads at the age of seven days.

Tests of reinforced concrete models of reduced scale

Henry Lossier, *Le Genie Civil*, Vol. CVI, No. 6, p. 138. Reviewed by R. L. Bertin.

The high cost of tests to destruction of full sized reinforced concrete structures leads frequently to the use of models constructed to a reduced scale. To appreciate

to what extent the results of the latter can be considered, as applicable to the structures themselves, it is necessary to consider two principal questions—the equality of the restraints and that of the resistance of the materials.

Equality of Restraints—For members of similar shapes, loaded equally per unit of surface, the bending moments are proportional to the third power of homologous dimensions. The same relation existing for the section modulus, the stress engendered by the bending moments is equal in corresponding parts of the full sized members and their models to a reduced scale. The same is true of shear stresses which, like the sections which resist them, assume values proportional to the second power of the homologous dimensions. The mathematical proofs of the above statements are given. In the case of concentric loads (P) the equality of restraints is realized when the loads P^1 of the reduced models are equal to Pn^2 in which n is the factor of reduction. For the weight of the structure itself, the restraints are proportional to (n).

Equality of Resistance of Materials—The author points out that the apparent elastic limit of reinforcing bars increases sensibly as their diameters decrease. Thus in test specimens where the reinforcing bars are the weak element, the bars used in the reduced models should have the same elastic limit as those of the similated structure irrespective of their size ratio. Concrete, on the other hand, in which the size of the aggregate is reduced shows considerable reduction in strength for the same cement proportion.

The author concludes that tests to failure on reduced models, in which all the elements are reduced in the same proportion, may lead to erroneous results, either too high or too low depending whether the steel in tension or concrete in compression cause failure, unless the characteristics of the materials in the structure and in their models are of the same order.

Long time tests of reinforced concrete beams with various grades of reinforcing steel

RUDOLF SALIGER, Bulletin No. 15, Austrian Reinforced Concrete Committee, 1935.
Reviewed by INGE LYSE.

This is a very timely contribution on the function of high grade steel in reinforced concrete beams. An excellent abstract of previous experiments, particularly fatigue experiments in European and American laboratories on reinforced concrete members, is presented as an introduction to the extensive fatigue investigation conducted at the Technical Institute of Vienna. The report gives detailed information on materials, manufacture of specimens, the test apparatus and the results obtained. Four different grades of reinforcing steel were used. Some of the most important results may be summarized as follows:

1. The strength of the concrete as determined on cubes varied from about 5500 to about 7000 p. s. i. The modulus of rupture was about one-seventh the compressive strength.

2. The yield-point strength of the reinforcing steel varied from about 41,000 to about 63,500 p. s. i. with percentage of reinforcement varying from 0.56 to 1.4 per cent.

3. The loads were applied at a rate of 160 to 170 repetitions per minute. The applied load was as much as 55 per cent of the maximum strength of the beam.

4. The first cracks appeared when the tensile stress exceeded the modulus of rupture of the concrete regardless of the grade of reinforcement used.

5. The failure of all beams was caused by tension failure of the reinforcement. The ultimate strength was not affected by repeated stresses up to 55 per cent of the

strength. The computed maximum stress in the reinforcement exceeded the yield-point stress of the steel used by about ten per cent.

6. The author recommends that permitted working stresses be set at 50 per cent of the yield-point stress of the reinforcing steel and 33 per cent of the ultimate strength of the concrete as determined on cubes. The 95-page report contains 55 figures and 32 tables.

Shrinkage of concrete at elevated temperatures, and heat conductivity of moist and dry concrete

Otto Graf, Bulletin 74, German Reinforced Concrete Committee, 1933. Reviewed by Inge Lyse

The shrinkage tests reported in this bulletin were made at temperatures of 18°, 60° and 120° C (65°, 140° and 248° F.). Effects of cement content, type of cement, and size of test specimen were studied at the different temperatures. The more important results may be summarized as follows:

1. Rapid heating of 7 by 7 by 17 cm. specimens and subsequent cooling produced shrinkage cracks.

2. For gradual heating and cooling only specimens of rich concrete mix showed cracks, the leaner mixes showing no cracks.

3. After fast heating to 60° and 120° F. the shrinkage was less than at room temperatures.

4. The richer the mix, the greater was the shrinkage.

5. The various portland cements gave essentially the same shrinkage which was in excess of the shrinkage for the high alumina cement.

The heat conductivity was determined on concrete plates. The results showed that:

1. The heat conductivity varied appreciably with the type of cement used.

2. The cement content did not have a consistent effect upon the heat conductivity.

3. The mortar content in the concrete did not affect the heat conductivity appreciably.

4. Concrete of high water content gave lower conductivity than concrete of low water content.

5. The type of aggregates used affected the conductivity.

6. The heat conductivity was greater at high than at low temperatures.

7. For dry concrete plates the conductivity generally decreased with the increase in voids.

8. The conductivity increased with increased moisture content in the concrete.

The test results are presented in tables and in diagrams.

Plastic flow of concrete under sustained stress

R. E. Davis, H. E. Davis and J. S. Hamilton, *Proceedings, Am. Soc. for Testing Materials*, Vol. 34, Part II, p. 354. Author's synopsis.

This paper describes laboratory tests made on plain concrete cylinders and reinforced concrete columns subjected to sustained compressive stress. The tests have been carried on in the Engineering Materials Laboratory of the University of California over a period of about seven years.

The test results show the influence upon plastic flow or time-yield of the following variables: (1) duration of loading, (2) moisture condition of storage, (3) moisture content of concrete, (4) temperature of storage, (5) gradation of aggregate, (6) mineral character of aggregate, (7) size of mass, (8) age at time of application of load, (9) intensity of sustained stress, (10) lateral restraint, and (11) reinforcement.

The extent of plastic recovery upon release of sustained load is indicated. In the studies on reinforced concrete columns, the variation in stress, with time, between the concrete and the steel is determined.

In the paper are discussed fundamental causes of flow, the interrelation between plastic flow and shrinkage, and the conditions under which flow under compressive stress may be important in design.

Among the conclusions it is pointed out that (a) flow in plain concrete may continue for periods of more than seven years, although the major portion of the deformation takes place within a year or two after application of load, (b) flow in reinforced concrete columns may practically cease after about one year under load for dry-stored columns, and after a few months for wet-stored columns, (c) the major portion of plastic flow is apparently due to seepage of colloidal water from the cement gel, although it is believed that some deformation may take place due to viscous and crystalline flow, (d) flow is markedly influenced by moisture condition of storage, moisture content of concrete, age of concrete, grading and character of aggregate, intensity of stress, size of mass, and reinforcement, (e) due to flow, the stress in the steel in reinforced concrete columns may in time reach or exceed the yield strength of the steel, while the concrete stress may be reduced to a value approaching zero.

International cement

STAFF ARTICLE, *Fortune*, Vol. XI, No. 3, p. 55.

"For a business that is just beginning to outlive its founders, the cement industry has already worked itself into a remarkably standardized condition. No portland cement was made in this country until 1872, when David O. Saylor began manufacturing it in the Lehigh Valley district, which was to be for many years the center of the cement industry. But there never would have been a U. S. cement industry if there had never been a U. S. cement tariff. Cement is a perfect case history for the special pleading of the protectionist because this country had raw materials in infinite abundance, and because cement, heavy and bulky, could be manufactured logically only in the same country that consumed it. Yet Belgian cement, profiting by low wages and cheap water transport, was being carried across the Atlantic Ocean. And selling at prices that discouraged U. S. capital from even trying to compete.

"Then (1890) along came Major William McKinley, who wrote a tariff bill that taxed foreign cement thirty cents a barrel. Up mushroomed a U. S. cement industry, growing so rapidly and yet so sturdily that in 1913, when Democrats removed the tariff, it still plunged along with amazing speed. (The tariff was restored again in 1930, at twenty-three, where it still remains.) Indeed, the International Cement Corp. with mills in Cuba, the Argentine, Brazil and Uruguay has nearly driven Belgian cement out of those countries. Thus the situation at McKinley's time has been beautifully reversed. Even after its tariff wall had been built, cement's troubles were not over. Everybody was making cement, but the product was so uncertain that roads or buildings made out of concrete were viewed with deep suspicion. Then, in the early 1900's—largely through the work of the Portland Cement Association—the cementmakers got together and thoroughly standardized their product in every detail." Thus for general comprehension does *Fortune* in one of its typical reviews go on for a dozen pages dramatising an industry and centering the action around the International Cement Corp. There are also some excellent pictures; for instance a shot at a cement kiln "the largest piece of moving machinery in any industry."

Effect of duration of moist curing on the principal properties of concrete

A. G. TIMMS, *Proceedings, Am. Soc. for Testing Materials*, Vol. 34, Part II, p. 329.
FROM AUTHOR'S INTRODUCTION and SUMMARY.

The literature on concrete contains a wealth of information on the effect of duration of moist curing on strength, but little information relating to its effect on such properties as durability and volume change. This investigation was undertaken to supply some of the needed information.

The data cover strength tests at from 1 day to 2 years of 6 by 12-in. concrete cylinders moist cured the entire time and part of the time; strength tests of 6-in. concrete cubes after different numbers of cycles of freezing and thawing; loss in weight of 6-in. concrete cubes subjected to freezing and thawing; tests of permeability by capillary flow; and length change and absorption tests on 3 by 3 by 15-in. mortar bars. The tests were conducted with a normal portland cement and two high-early-strength portland cements.

Following are the principal results:

1. Typical water-cement ratio-strength curves for moist-cured concrete were obtained regardless of the type of cement or test age. A significant feature of the strength tests was the smaller differences between the strength of moist-cured and air-cured specimens at ages greater than 28 days exhibited by the high-early-strength cement.

2. Increased expansion of mortars in moist air but decreased shrinkage in dry air at 18 months resulted from longer periods of moist curing. This was true for all water contents and for both types of cement. The difference in volume change between the normal portland and high-early-strength cement was not of great importance.

3. Mortars of better quality were less absorptive, regardless of whether the better quality was due to a lower water content or prolonged curing.

4. Up to a certain period, increasing the duration of moist curing of the concrete before exposure increased its resistance to the destructive effects of freezing and thawing, but further curing had no effect in increasing resistance. The length of this beneficial curing period depended on the type of cement and water content of the paste. With a given cement and curing periods sufficient to develop the maximum resistance in each case, the richer mixes gave greater resistance than lean mixes of the same strength at the time of exposure.

5. The rate of capillary flow in mortars is influenced by duration of exposure, by type of cement and quantity of mixing water. Even the mortars of high water content in a relatively short time reached a uniform rate of leakage which was quite small as compared with the rate of leakage when first exposed.

Pressure of concrete on forms

HARRISON G. ROBY, *Civil Engineering*, Vol. 5, No. 3, Mar. 1935, p. 162. Reviewed by J. R. SHANK.

Tests were made on four different concrete mixes under four different rates of placing with six different temperatures to find the amounts and durations of pressures and at what heads of green concretes these occur.

The apparatus consisted of a 2 ft. 6 in. square wood column form with a steel plate at the bottom of such size and so placed as to have a horizontal deflection characteristic equivalent to the column form piece it displaced. The deflection of this plate was measured by means of a lever and a scale.

The mixes used were *Normal, Dry, Rich* and *Lean;* 1:2:3½, 1:2:3½, 1:1¼:2¼ and 1:2½:5; having slumps of 7, 3, 7 and 7 in.; cement-water ratios by weight of 1.65, 1.75, 2.27 and 1.36; and weights per cu. ft. of 151, 151, 150 and 154 lb. The aggre-

gates used were sand and gravel 0 to 4 mesh and 8 mesh to 1½ in., having weights per cu. ft. of 99 and 103 lb.; and fineness moduli of 2.86 and 7.38.

The rates of placing were 1 ft. per hr., 2, 4 and 10 ft. per hour. The temperatures of the concretes when placed were 50, 60, 70, 80, 90 and 100° F. The maximum pressure obtained was 1000 lb. per sq. ft. for the rich mix under a head of 8 ft. at 100 mins. after placement was started, for the 4 ft. per hour rate of placement. The normal mix reached its maximum pressure 780 lb. per sq. ft. for the 10 ft. per hour rate of placement at the 6 ft. head. The lean and dry mixes both reached their maximums at 5 ft. head. The maximum pressures for the lean and dry mixes were 630 and 580 lb. per sq. ft. respectively.

Quoting from the author:

"1. For concretes of normal mix and at moderate temperatures, a rate of placing of 10 ft. per hour will develop a pressure of about 800 lb. per sq. ft., a rate of 4 ft. per hour, that of about 700 lb. per sq. ft., and a rate of 2 ft. per hour, that of about 600 lb. per sq. ft.

"2. An extremely rich concrete will develop much higher pressures than a normal mix. This increase in pressure varies from about 40 to 60 per cent, the lower percentage being for the more rapid rate of placing.

"3. A lean mix develops pressures from 10 to 15 per cent less than a normal mix.

"4. A dry mix develops pressures from about 20 to 25 per cent less than a normal mix.

"5. Pressures developed during hot weather are less than those during moderate temperatures. Mixtures having a temperature of 100° F. when placed will develop pressures equal to from 60 to 75 per cent of those developed by the same mixes at 60° F.

"Immediately after the maximum pressure was reached the bottom form boards could be removed without having the concrete fall."

Laboratory and field tests of Bayshore Highway cooperative investigation

T. E. STANTON and RAYMOND E. DAVIS, Mimeographed report to cooperating agencies, Oct. 1933, 75 pp. Reviewed by J. W. KELLY

The report covers laboratory and field tests on concretes containing four standard, five high-early-strength, and three special cements employed in special test sections of the Bayshore Highway near San Francisco. The field tests included observations of volume changes and temperatures throughout the depth of small test slabs, manufactured and cured under conditions closely similar to those for the highway. Supplementing the actual test results, the report includes a critical analysis of volume changes and stresses within highway slabs.

The authors analyze the separate and combined effects of temperature and moisture upon the daily, seasonal, and permanent movements, which consist of (a) changes in length of slab as a whole, (b) curling of slab, and (c) internal stresses and surface cracking.

The laboratory and field tests indicated that among the twelve cements there were no marked differences as to strength, volume changes, or resistance to weathering simulated by alternations of freezing and thawing and alternations of wetting and drying. It was found that the tendency to curl due to unequal moisture contents and temperatures at top and bottom of slabs was of greater importance than the volume changes of the slab as a whole.

Due to the heat generated by the hydrating cement, the temperature of the concrete in the pavement slabs rose approximately 20° F., the amount differing with the cement. For a given cement, the concrete cast near noon attained a higher maximum temperature than concrete cast during either the morning or the afternoon. There were indications that casting concrete at night might prove advantageous for pavement slabs.

Although the tests were carried out under mild climatic conditions, it was found that the tendency toward curling was large. The daily curling of slabs due to temperature change was sufficient to produce an alternate rise and fall of the ends of an unrestrained 20-ft. slab of $\frac{1}{16}$ in. downward during the day and $\frac{1}{16}$ in. upward during the night. The curling of slabs due to the difference in moisture content between top and bottom of the slabs up to the age of 1 year was sufficient to cause the ends of an unrestrained 20-ft. slab to rise $\frac{5}{16}$ in. Within the first year, the contraction of the slab as a whole, due to moisture change, was only 50 per cent of that due to temperature change; but the curling of the slab due to moisture change was five times that due to temperature change.

Daily variations in temperature of field test slabs were greater for cements of darker color. Slabs differing in natural color differed by as much as 8° F. in surface temperature when the sun was shining.

Laboratory measurements of contraction due to drying satisfactorily indicated the relative *tendency* of various concretes to contract in the field. However, the *actual* amount of contraction in the field was influenced by the amount of surface cracking.

The test results indicated that factors other than the cements themselves might be varied to control the curling and volume changes of concrete slabs. The authors suggest the desirability of further investigations to determine the effect of such factors as type of aggregate, surface coatings, and variations in thickness of slab.

The new reinforced concrete bridge across the Seine at La Roche-Guyon (Seine-et-Oise), France

S. Boussiron, *Le Genie Civil*, Vol. CVI, No. 6, p. 125-132 and N. Esquillan, *Le Genie Civil*, Vol. CVI, No. 7, p. 155-159. Reviewed by R. L. Bertin.

This bridge consists of a main arch span of 161 meters and approaches totaling 201.8 meters over all. The rise of the arch is 23 m. or 1/7 of the span. The deck is suspended from the two reinforced concrete ribs, spaced 9.40 m. on centers, by hangers 7.90 m. apart, consisting of 8 bars encased in an octagonal concrete shaft .25 m. over all. The deck is 10 m. wide between railings; roadway 5.60 m., and two sidewalks 2.2 m. The deck crosses the ribs 7.50 m. above the spring line, the main ribs being 14.55 m. above the deck elevation at mid span.

The ribs are integral with the abutments; their depth increases gradually from 1.45 m. at the spring line to 2.65 m. at the crown; their width is 1.40 m. above the deck and gradually increases to 3 m. below it. The ribs are in the form of a rectangular hollow box, the top and bottom slabs being .52 m. thick and the vertical sides .14 m. As the arches reduce in thickness, the top and bottom slab approach each other and join in a solid slab 1.00 m. thick where the deck crosses the arches. They separate again, forming the top and bottom of a rectangular box, widening gradually to 3 m. in width where they join the abutments.

The moment of inertia is so varied along the arch as to provide maximum flexibility and makes the maximum stresses practically constant along the entire arch. Graphs are given showing the influence of this design on the moment distribution and working stresses as compared with arches of constant sections and those in which the moment of inertia increases from the spring line towards the crown. The ribs are reinforced

with small spirals coiled to a diameter of .20 to .25 m. nested together, the longitudinal reinforcement being used only to hold the ties and spirals in line and is not depended upon as a medium of resistance. The adopted concrete working stress was 90 kg per sq. cm.

The abutments are carried on concrete piles part of which are vertical and the rest inclined at 45 deg. The thrust of the arch is resisted partly by the inclined piles and partly by earth resistance back of the abutment and back of a retaining wall rising from the natural soil to the deck, this resistance being transmitted to the abutments by means of a ribbed reinforced concrete slab.

The supports for the form work consisted of three towers, one at mid-stream and one on each side 43.48 m. from the center one, all supported on piles. A flat deck was erected from shore to shore, suspended from the towers by means of cables. On top of this deck, the forms for the arches were erected, consisting of a solid deck of double thickness of planking at 45 deg. to each other, following the intrados of the arches supported on timbers.

The upper deck was linked by means of cables to the lower deck so as to form a parabolic truss from shore to shore of great rigidity. The top decking was prestressed by means of jacks to stabilize it. The arches were concreted on top of this decking in parts; first the bottom slabs were poured from the spring line towards the crown, allowed to set for 7 days and prestressed by means of jacks. The slab was anchored vertically and laterally to the forms to prevent buckling but allowed freedom of movement longitudinally. Sufficient pressure was applied to relieve the forms of the weight of the concrete slab. The sides of the ribs were then cast and additional jacks placed between the sections and pressure applied. The top slabs were then poured, thus completing the arches. By manipulating the jack pressures, the thrusts and moments were controlled. The decking was released from the arches by removing the jacks which held it in place. The jacks at the crown of the arches were kept in place until after the bridge had been tested and all foundation yields as indicated by settlement of the arches had stopped.

As a test load, the form work, the weight of which corresponded to the required test load was suspended from the arches and for unsymmetrical loading the forms over one-half of the span were removed; the rest allowed to remain until all readings had been taken.

Interesting data are given showing the possibility of much larger spans of similar construction from the standpoint of design and execution. The determination of the modulus of elasticity of the concrete from the pressures exerted by the jacks over the observed displacements is treated at length.

Construction Features of the Zeiss Dywidag Dome for the Hayden Planetarium Building*

BY R. L. BERTIN†

MEMBER AMERICAN CONCRETE INSTITUTE

A PLANETARIUM is an optical apparatus specially constructed for the purpose of illustrating the relative motions of celestial bodies.

The observation room is provided with a hemispherical ceiling, from the center of which the planetarium is made to project thereon images of the heavenly bodies in their accurate relative positions and are made to move across the ceiling, representing the sky, in their true course. The apparatus is so devised that the relative position and motion of the stars and planets may be shown for any time through the ages as viewed from any part of the earth. The demonstrations are explained to the audience by a lecturer.

A number of these exhibition buildings, popularly known as planetariums have been built in Europe since the improvement of the projectors by the Carl Zeiss Co. of Germany, manufacturers of this intricate apparatus.

Planetarium buildings are gaining in popularity in this country. One in Philadelphia and one in Chicago have been in operation for some time; one in Los Angeles is nearing completion; the fourth is the Hayden Planetarium Building now being constructed by the White Construction Co., Inc., for the American Museum of Natural History in New York.

The building is a two story and basement structure of the structural steel skeleton type with cinder concrete arches and brick facing with stone and terra cotta trimmings, surmounted by a reinforced concrete dome which is made a feature of the architectural design.

The projection surface consists of a sub-dome, hung from the main dome, constructed of Johns-Manville acoustical perforated stainless steel sheets. Its diameter is 5 ft. smaller than that of the main dome and its center 7 ft. 2 in. lower, thus providing a space between the two domes in which ventilating and sound equipment is provided and made accessible by means of catwalks and ladders.

*Presented at the 31st Annual Convention, American Concrete Institute, New York, Feb. 19-21, 1935.

†Chief Engineer, White Construction Co., Inc., New York.

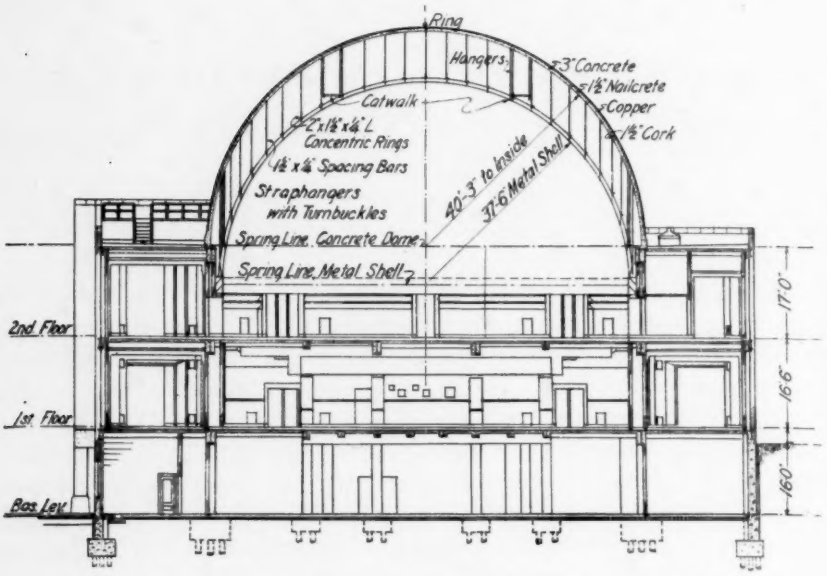


FIG. 1—CROSS SECTION OF HAYDEN PLANETARIUM

To eliminate sound reverberation, the under side of the main dome is lined with rock cork 1½ in. thick.

Fig. 1 shows a cross section of the building showing the relation of the different parts.

The plans and specifications prepared by Trowbridge and Livingston, Architects, and their structural engineers, Weiskopf and Pickworth, provided for a reinforced concrete shell dome similar in construction to those used in Europe. An alternate bid was asked for a Gustavino tile dome, in lieu of the concrete dome. The White Construction Co., Inc., the low bidders for the general contract, estimated the cost of the concrete shell dome to be lower than the price quoted by the tile dome constructors. The architects, accordingly, selected the concrete dome. The actual cost of constructing the concrete dome verified the judgment of the White Construction Co., Inc., as to the relative cost of the two types of construction.

Before describing the construction of the dome which is the subject of this paper, a brief history of the evolution of curved structures from the early ages to date is given as a matter of interest.

Dome construction is one of the oldest in the building art. The Pantheon dome with a span of 144 ft., built by the Romans; the 103 ft. dome of the St. Sophia Mosque in Constantinople, built in the sixth

century by Emperor Justinian; the 138 ft. dome in Florence, Italy, built in the fifteenth century; the 131 ft. dome of St. Peter in Rome, built in the sixteenth century, and later a number of domes executed bythe Turks are remarkable examples of curved structures.

These masterpieces of massive dome construction were not surpassed until 1913 when the Market Hall of Breslau was built with a span of 213 ft.

Nothing is definitely known of the knowledge of dome designs in the ancient periods. However, the actual execution of structures of this type prove that the old masterbuilders were acquainted with the theory of thrust lines and its interaction on ring stresses. The rather vague and crude knowledge of structural action and the lack of material of high tensile strength necessarily led to structures of very heavy dimensions.

Since the construction of the Pantheon in Rome, no essential progress in the theoretical and constructive art of dome construction was made until the first theories on domes of rotation for symmetrical loading, known as the membrane theory, were developed by G. Lamé and E. Clapeyron in the year 1828.

The invention of reinforced concrete resulted at the beginning of this century in the revival of dome construction. However, the lack of knowledge regarding the action of shell slabs along their open edges and the magnitude and location of bending moments acting transversely to the shell slabs was responsible for the adoption of rib domes of massive proportions, of which the Breslau Market Hall is an outstanding example.

In a rib dome, comparatively little use is made of the distribution of forces in space. The dead and external loads are transferred chiefly by bending action of the rib in its own plane.

Shells are structural elements with singly or doubly curved surfaces; usually slabs or plates very thin in comparison to the surface area. The statical action of these structures is in three dimensions and, as a consequence, actual principal stresses are considerably smaller than in structural systems the action of which is in one plane only.

It was not until the Zeiss Company and its affiliated company, Dyckerhoff & Widmann of Germany began developing and their engineers solved the analysis of shell structures that the construction of domes with shells as thin as 1½ in. could be built.

The cooperative work of the firms of Carl Zeiss and Dyckerhoff & Widmann led to the development of shell structures of various types, not only for churches, planetariums and the like, but they turned their attention to the construction of roofs of large spans for industrial

structures. Doubly curved shells were first considered but while they were ideally suited for the transfer of loads by direct stresses to the supports, they could not be solved owing to complication in the solution of the differential equations involved. Ultimately, a system of singly curved shells, stiffened at their open ends by diaphragms, was evolved in which the internal stresses could be determined.

A few illustrations of shell roofs (Fig. 2 to Fig. 5 inclusive), show the possibilities of this type of construction as applied to structures where exceptionally large spans and lofty ceilings are required, as for instance—market halls, hangars, pier sheds and the like.

These remarkable structures, known as Zeiss Dywidag shell roofs, a considerable number of which have been built in Europe, are now being introduced in this country.

Coming back to the description of the Zeiss Dywidag dome for the Hayden Planetarium; the reinforced concrete shell dome is a hemisphere, 80 ft. 6 in. in diameter on the inside; the shell is 3 in. thick except for the first 9 ft. from the spring line where the thickness is 3½ in. A stiffening ring 12 in. thick and 18 in. high forms the base of the dome which bears on the structural steel, primarily at 8 points consisting of steel beams cantilevered approximately 36 in. from the carrying columns. The ring rests also on steel beams framing between the columns. Because of the extreme rigidity of the dome structure, the distribution of the load between the cantilevers and the in-between beams is a function of their relative deflections.

The writer is informed that the dome, as designed, meets all requirements for domes supported on continuous rigid supports, and for domes supported on point supports as determined by the theories of design developed by Carl Zeiss.

The reinforcement of the shell consists of a network of ¼ in. and ⅜ in. bars in the form of horizontal rings and meridional bars, the spacing of which varies from 4 to 6 in.

The specifications call for the concrete bottom ring to be poured and the balance of the dome to be gunited, with a mixture of one part of cement and three parts of sand, capable of developing a strength of 2000 p.s.i. at four days and 3500 p.s.i. at 28 days.

The specification for the forms of the structure placed the responsibility on the contractor to design and construct them so that at no point would the surface deviate from a true hemisphere by more than ½ in.

In selecting a suitable method of constructing the form work, the usual procedure of constructing them on shores was deemed inadvis-

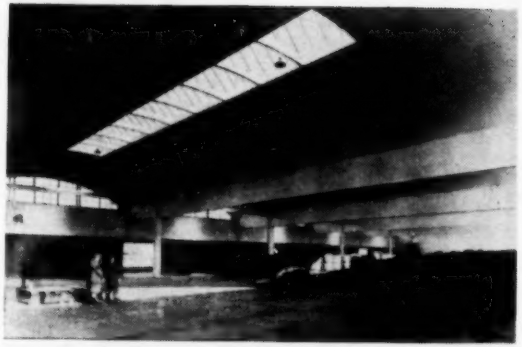


FIG. 2—WAREHOUSE IN JUGOSLAVIA

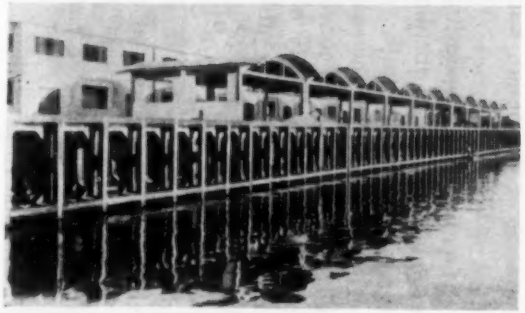


FIG. 4—DOCK SHED IN SPAIN SHOWING USE OF OPEN BARRELS

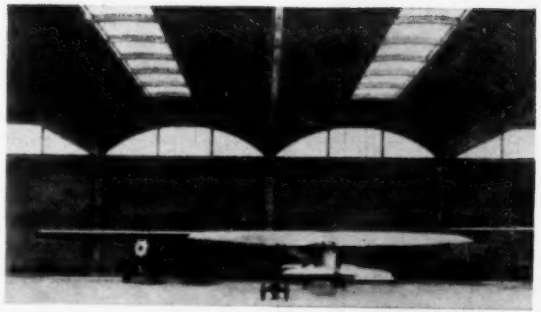


FIG. 3—HANGAR IN ITALY

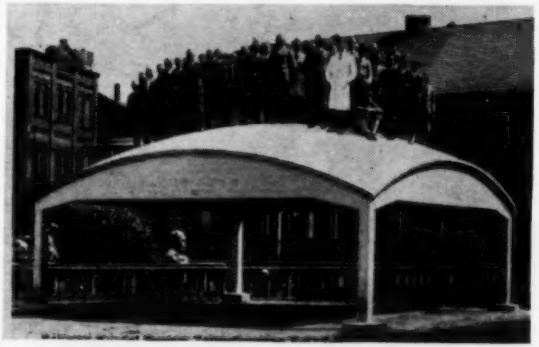


FIG. 5—TEST STRUCTURE SPAN 24 X 24 FT. SHELL THICKNESS $\frac{5}{8}$ IN.

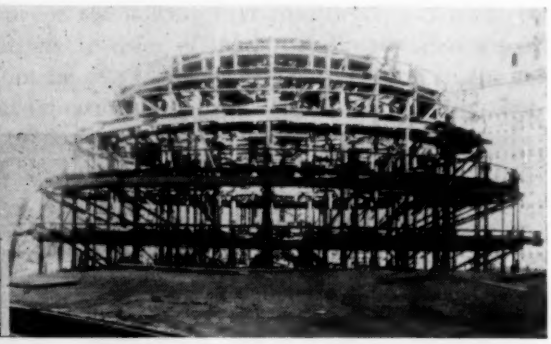


FIG. 6—(LEFT) SLOTTED PIPE SLEEVES CONNECTING THE SECTIONS OF PIPE RINGS

FIG. 7—(RIGHT) WORKING SCAFFOLD FOR INSTALLATION OF FORM WORK

able owing to the uncertainty and difficulty involved in locating and maintaining accurately every point of a space structure within the tolerance specified. A system therefore was sought which would tend to assume a spherical shape. This led to the adoption of a self supporting structure, the behavior of which would follow the geometrical characteristics of a sphere.

The development of the design was subordinated to the following requirements:

(1) Ease of erection and dismantling.
(2) Dimensional accuracy of the parts.
(3) Positive jointing of the units.
(4) Strength and rigidity of the assembly.

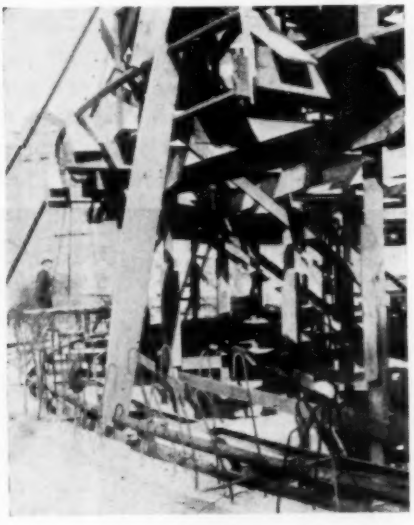
The design finally adopted consisted of shop fabricated units assembled in place. The assembly was made up of three elements: 1st—8 horizontal pipe rings, 2nd—about 1000 meridional wood ribs, and 3rd—about 5000 board feet of sheathing in the form of $\frac{13}{16}$ in. x 4 in. strips.

The meridional ribs were cut in a mill from 2 x 10 in. rough spruce planks; one edge was cut to the curvature of the dome and two slotted holes, one at each end, were drilled the proper distance apart, $\frac{1}{8}$ in. larger in diameter than that of the pipe used for the rings.

The horizontal pipe rings were spaced at equal distances and consisted of x. h. pipe $2\frac{1}{2}$ in. nominal diameter, weighing 7.6 lb. per lin. ft. Stock lengths 20 to 22 ft. long were bent accurately to the proper radius. Slotted sleeves were welded to one end of each section of pipe. The sections were then assembled and fitted in the shop to

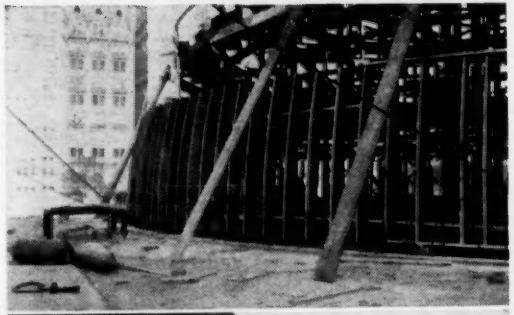
FIG. 8—VIEW OF ANCHOR RODS AND STRUCTURAL CHANNELS CARRYING THE FIRST HORIZONTAL PIPE RINGS

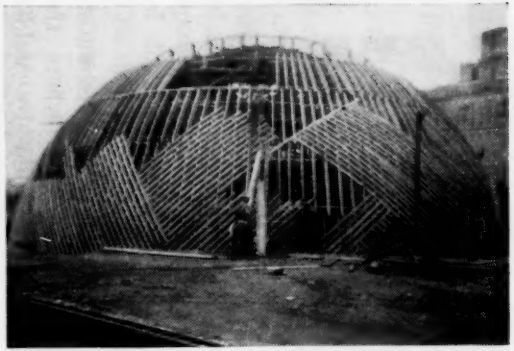
FIG. 9—HORIZONTAL PIPE RINGS AND MERIDIAN RIBS



a complete ring and tested for correct diameter and curvature. The sections were marked and shipped to the job where they were reassembled in place and the splices completed by welding the open ends of the sleeves to the pipe as shown in Fig. 6.

A working scaffold was erected as shown in Fig. 7, the posts were located directly under the pipe rings and spaced in radial bents 15 ft. apart. The distance between posts did not exceed 10 ft. Working circular platforms were erected about 2 ft. below the level of each ring. The posts were capped with a T-head, located a few inches below the final elevation of the pipe rings, on top of which the sections of the pipe rings were assembled. Nails driven in the T-heads in line with the theoretical positions of the outer face of the pipe rings served as gauges to check the diameter and centric positions before the sections were finally welded into a continuous ring. This process was repeated for every ring except the first one which rested on small channels previously installed which were to serve as permanent supports for the cat walk at the spring line, Fig. 8. The rings were set at elevations approximately ¾ in. lower than their theoretical one.





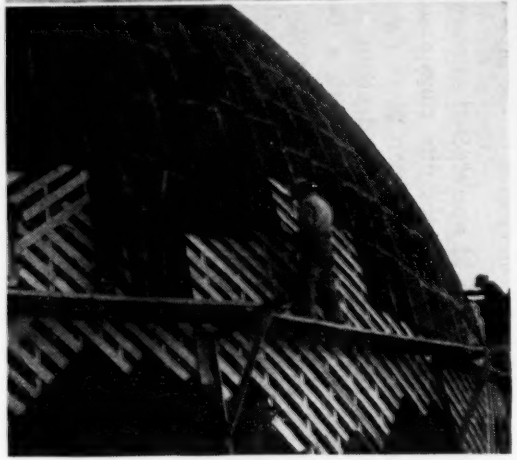


FIG. 10 — (TOP LEFT) FIRST ROW OF MERIDIAN RIBS

FIG. 11—(TOP RIGHT) GENERAL VIEW OF THE FORM WORK DURING ERECTION

FIG. 12—(BOTTOM LEFT) INSTALLATION OF ROCK CORK ON TOP OF THE LATTICED SHEATHING

FIG. 13—(BOTT. RIGHT) ROCK CORK COVERING COMPLETED AND FORMS FOR CONCRETE EDGE RING IN PLACE

After the rings were all set and welded, the meridional ribs were put in place, starting from the bottom. The lower hole of the rib was slipped over the lower pipe and the rib slanted sufficiently for the upper hole to engage the upper pipe. The rib was then hammered to a vertical position, (Fig. 9). As the ribs were progressively installed, about 16 in. apart, the upper ring was gradually lifted off the T-heads until when all the ribs were in place this ring was entirely free from the scaffolding (Fig. 10). The elevation of the ring was then checked all around and minor adjustments made until the ring was horizontal.

The sheathing was then started laid diagonally across the ribs in order to brace the assembly, as shown in Fig. 11.

The 4-in. wide sheathing was laid 4 in. apart, the rock cork being depended upon to form a continuous surface to receive the concrete (Fig. 12).

The process was repeated from ring to ring until when the last ribs were in place the entire form was free from the scaffold and remained in that condition until the dome was completed, Fig. 13.

The inner surface of the forms was checked from the center of the dome during their construction and during the concreting. No movement of any consequence could be detected resulting from the weight of the gunite or wind which at times was of gale magnitude. Check on the finished dome indicated that the structure is within the allowable tolerance.

The use of pipe rings in combination with the drilled meridional ribs furnished positive hinged joints requiring no fitting in the field.

While theoretically this form of assembly does not offer any resistance against lateral or torsional movement of the upper ring with reference to the lower one, the frictional resistance of the ribs on the pipe rings was sufficient to hold the rings in place before the sheathing was applied.

The diagonal sheathing was relied upon to provide the necessary bracing against displacement. It proved very efficient, as evidenced by the lack of movement during wind storms or the unsymmetrical pressures resulting from the application of the gunite on one side.

In proportioning the rings and ribs, it was assumed that the dome would be called upon to carry a zone of concrete approximately 8 ft. wide at a time in addition to the dead load of the form work. For the sake of simplicity, all rings were made of the same size pipe and the meridional ribs all alike.

Owing to unforseen foundation condition, necessitating a revision of the original design, the construction of the structure was

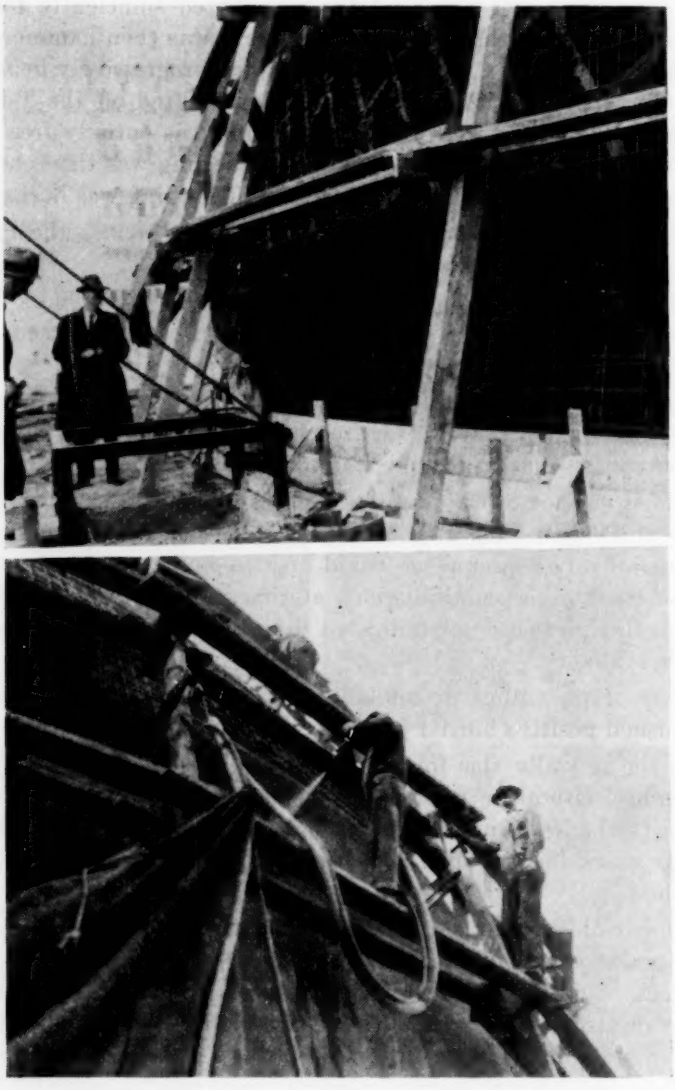


FIG. 14—INSTALLATION OF REINFORCEMENT
FIG. 15—OPERATION OF CEMENT GUN

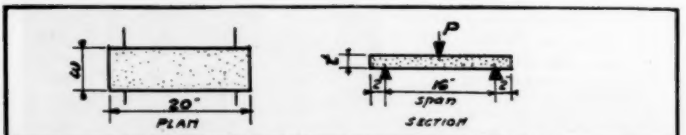
FIG. 16—GENERAL VIEW OF DOME DURING APPLICATION OF GUNITE

delayed and brought the dome construction at a time when cold weather and sudden drops of temperature might be expected. It was deemed prudent to use a high early strength cement. The materials were heated and protected by means of tarpaulins draped over the day's work.

The concrete ring at the bottom was poured in one continuous operation and the gunite was applied in zones of such height as to enable their completion in one day. It required 11 working days to complete the guniting; Fig. 14 shows the reinforcing rods being installed; Fig. 15 shows the gunite being applied; Fig. 16 is a general view of the dome during construction.

For each zone of gunite, test slabs were made 1¾ in. thick, 6 in. wide and 20 in. long. The slabs were given as near as possible the same curing conditions as the concrete in the structure. They were tested by the Pittsburgh Testing Laboratory at various ages with a center load on 16 in. span. The results of the tests are given in Table 1, Fig. 17.

The test specimens are too few in number and the conditions under which they were cured were too varying to warrant any conclusion, except that none of the specimens shows a modulus of rupture below that corresponding to a crushing strength of at least 3000 p.s.i. at four days.



Specimen Mark	Date cast	Date Tested	Age Days	"w" inches	"t" inches	S.M $\frac{wt^2}{6}$	P	Modulus of Rupture	Probable Comp. strength ◆
A	12/4	12/6	2	6.0	1.75	3.06	550	720	3900
B	12/4	12/18	14	6.0	1.75	-	640	837	5350
C	12/5	12/19	14	6.1	1.80	3.30	590	715	3850
D	12/5	12/19	14	6.1	1.80	-	690	837	5350
E	12/7	12/21	14	6.1	1.75	3.12	800	1025	8000 +
F	12/13	12/17	4	6.0	1.75	3.06	500	653	3250
G	12/13	12/27	14	6.0	1.75	-	630	824	5150
H	12/13	12/27	14	6.0	1.75	-	770	1009	8000 +

Mix.. Cement (H.E.S.) 5 cub. ft
Sand 10.5 - -
¼ Grit 7.0 - -
Mixture applied by cement gun hydrated at nozzle.

◆ Relation of Mod. of rupture to Comp. strength taken from "Lone Star research laboratory" Diagram MR-3
+ These specimen are beyond the range of the diagram.

RESULTS OF FLEXURAL TESTS ON SPECIMEN OF GUNITE.

FIG. 17—RESULTS OF FLEXURAL TESTS ON SPECIMEN OF GUNITE

After the last section of gunite was placed and the test slabs indicated a modulus of rupture corresponding to 3000 lb. compressive strength, the stripping of the forms was started.

All pipe rings were wedged onto the T-heads and the wedges supporting the lower ring removed, and the ring dropped.

The first row of ribs was then removed, the second ring was then freed from the T-heads and dropped and the ribs removed. This process was continued to the top.

On top of the concrete shell, a coating of 1½ in. thick of nail code was plastered on to form a nailing base for the copper covering which forms the finished surface of the dome. As gauges and anchors for the plaster coat, double headed nails with the heads 1½ in. apart were driven in the gunite before it was set.

After completion, and before the copper roof was in place, the dome was subjected to several very heavy rains, one lasting 24 hours. No signs of water or moisture could be found on the inside, indicating dense concrete, perfect bond between the successive zones, and freedom from cracks.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by Aug. 1, 1935.*

Concrete In The Construction Of Dwellings

Editor's Note—*At the 31st Annual Convention in New York, Feb. 19-21, 1935, an entire evening session was devoted, not to a review of the use of concrete in house construction but to recent developments in such use, a handling of the subject by four speakers which gave it currency and timeliness. The four papers follow. Mr. Zipprodt covered a great deal of ground, briefly citing recent examples of very diverse use of the material in and near New York. Messrs. Newmark and Copeland presented the results of a recent investigation of monolithic walls for residences. Mr. Kaiser, with his several collaborators, presented the situation of the concrete joist floor, its aspects in use and from the results of recent tests. Mr. Earley told how his concrete mosaic offers its special solution of the housing problem. To those readers who have followed the technical developments of the Earley concrete mosaic through his papers to the Institute over a number of years, it will be interesting that since the New York convention he has been persuaded to record for the Institute an account of adaptations of mosaic slabs in the new United States Department of Justice Building, including developments which lie between the Baha'i Temple (March-April 1934* Journal*) and the small house work. This short paper is therefore published also in this* Journal.

Recent Noteworthy Developments in Concrete's Use in Housing Construction*

BY R. R. ZIPPRODT†

MEMBER AMERICAN CONCRETE INSTITUTE

The recent activities of the United States government, particularly those of the Housing Division of the Public Works Administration, the Subsistence Homesteads Division of the Department of the Interior, and the Federal Housing Administration, have tended to concentrate the attention of the public on the desirability of engaging in some form of housing construction. The Federal government, through the provisions of Title I of the Federal Housing Act initiated a large amount of rehabilitation and modernization construction, the volume of which, according to reports recently issued, is mounting rapidly. According to a statement made by the Federal Housing Administration just a week ago, $250,000,000 worth of modernization work has been done to date. However, the Federal government recognizing the fact that there is a shortage of housing units—variously reported to be anywhere from 150,000 to 800,000, and the Federal Housing Administrator has estimated as high as 1,500,000 units—and that the return of this country to some degree of normalcy is dependent upon when and how the construction industry recovers, is now making it possible to engage in the construction of single or multiple family units through the application of Title II of the Federal Housing Act.

In the eight northeastern states—the territory which is covered by the writer in the conduct of his work—there appears to have been developed a larger and more general interest than elsewhere, in the construction of single-family homes. Many of the houses so constructed have embodied some phase of fire-resistive construction or have been built of thoroughly fireproof construction throughout.

In reply to a questionnaire recently sent out by the New York Office of the Portland Cement Association pertaining to a competition for

*Presented at the 31st Annual Convention, American Concrete Institute, New York, Feb. 19-21, 1935.

†Regional Structural Engineer, Portland Cement Association, New York, N. Y.

FIG. 1—RESIDENCE FOR MISS GRACE DEVINE, COPIAGUE, L. I., NEW YORK. C. F. ROSBORG, ARCHITECT. COST LESS THAN $3200 EXCLUSIVE OF LAND

the construction, by Oct. 31, 1935, of fireproof homes in the New York metropolitan area, there were received more than 4,000 replies, indicating either interest in, or actual plans for the construction of such homes. Since late in 1933, 58 homes of this character have either been completed or, are nearing completion in the area under discussion.

For the purposes of discussion, the houses so constructed will be classified as of low-cost, medium cost or high cost. It is in the low-cost field, however, where architects have vied with each other to see which one could produce the most economical fire-resistive or fireproof house; one which would, at the same time, be thoroughly attractive and home-like.

One of the architects in the metropolitan New York area who has succeeded in accomplishing this purpose to a remarkable degree is C. F. Rosborg, who already has two residences in this classification to his credit. One of these—the residence for Miss Grace Devine at Copiague, Long Island, now under construction has a full basement, three rooms and a bath on the first floor and a bedroom on the second floor. Footings are of stone concrete, while the foundations and exterior walls are of 8-in. cinder concrete masonry units, painted one coat of white paint on the exterior. Three coats of plaster have been applied to the interior surfaces, placed over a paper-backed, ungalvanized wire fabric. Floor and roof construction are of wood. All sheet

FIG. 2—RESIDENCE FOR CHARLES J. SCHMIDT, LYNBROOK, L. I., NEW YORK. C. F. ROSBORG, ARCHITECT. COST LESS THAN $5000 EXCLUSIVE OF LAND

metal work is of copper. The significant point about this residence is that its cost, including a one-pipe steam heating system, plumbing, electrical installation including fixtures and finish hardware was only $3,200, exclusive of land cost. This is a cost of only 23 cents per cu. ft.

Another of Mr. Rosborg's residences is that for Charles J. Schmidt at Lynbrook, Long Island. This house, containing five rooms, a laundry room and a one-car garage, has only sufficient basement to provide for the oil-burning, steam heating plant, and to afford access

FIG. 3—PLAN OF CHARLES J. SCHMIDT RESIDENCE, LYNBROOK

FIG. 4—RESIDENCE FOR MARTHA PEABODY, WESTPORT, CONN. FRANK HARPER BISSELL, ARCHITECT. COST $2500

to the heating, water-supply and drainage lines. Over this basement there is a 5-in. stone concrete floor slab. Foundation walls are of 8-in. sand-concrete masonry units. Exterior walls, above the foundation are of 8-in. cinder concrete masonry units, furred with 1-in. cinder concrete furring blocks, 8 by 16 inches. Interior partitions are of 4-in. cinder concrete masonry units. All pitched roofs are covered with black Bangor slate; flat roofs with four-ply slag roofing. The steel casement sash are glazed throughout with double strength glass, the windows being set in the masonry openings and caulked with a waterproof mastic. Precast cinder concrete lintels were used over all openings in both exterior walls and interior partitions. Window sills are of cinder masonry units, finished with cement. The exterior walls are finished with two spray coats of white portland cement paint, and interior walls and partitions with two spray coats of similar paint, varying in color, depending upon the room in which used. The cost of this house, including finish hardware, electrical work and lighting fixtures, plumbing and a one-pipe steam heating oil-burner system was less than $5,000, exclusive of the land, or a cost of 28 cents per cu. ft.

Another architect, whose work has attracted national attention, and been featured extensively by the Woman's Home Companion, is

FIG. 5—PRECAST CONCRETE JOIST AND FLOOR SLAB CONSTRUCTION AS DEVELOPED BY BEDFORD HILLS CONCRETE PRODUCTS CORP.

Frank Harper Bissell. His one and one-half story house, built for Martha Peabody, near Westport, Conn., involved novel features, both of design and construction. The house has two rooms on the first floor and two on the second floor, and is of random ashlar cinder-concrete masonry units 8 in. thick, to the exterior of which two coats of white portland cement paint have been applied. Walls are unfurred and have been painted directly on the interior with two coats of portland cement paint. Floors are constructed of a system of precast concrete joists and slabs, as developed by the Bedford Hills Concrete Products Corp., Bedford Hills, N. Y. The cost of the house was $2,500.

Another fine example of a low-cost residence is that of the Bedford Hills Concrete Products Corp., Bedford Hills, N. Y., in which the exterior walls are of 8-in. concrete masonry units of random ashlar pattern, colored by the use of mineral oxides. The floors are of a system of precast concrete joists of novel design, combined with 3-in. precast cinder concrete slabs, all of which are grouted in place with a 1- or 1½ in. layer of 1:3 cement grout. Stair treads and risers are of precast concrete. The roof is framed in wood and covered with a concrete tile, colored to simulate tile considerably weathered by exposure to the elements.

The house of Loretta Sonnebend at Brookline, Mass., designed and built under the supervision of Alexander T. Saxe, Architect, of New

FIG. 6—RESIDENCE FOR LORETTA SONNEBEND, BROOKLINE, MASS. ALEXANDER T. SAXE, ARCHITECT

York, is also in the low-cost classification. It has five rooms and one bath; floor construction consists of junior beams with a 2-in. concrete top slab placed over self-furring metal lath. Exterior walls are of 8-in. cinder concrete masonry units, painted two coats of buff portland cement. The cost of this house, leaving the two rooms on the second floor unfinished, was $6,469, or 18¢ per cu. ft. This extremely low cost was accomplished with the architect acting as his own contractor, and sub-letting all items, rather than awarding construction of the house to a general contractor.

The two-story type of residence which has been developed by Inter Industries, Inc. of Rochester, N. Y., and built at Brighton, N. Y., with C. Storrs Barrows as architect, also merits serious study. Here we have a six-room, two-story house, 21 by 34 ft. with a two-car garage 18 x 18 ft. adjoining the house in the rear. The ground floor is of concrete, laid on fill; the second floor and second story ceiling are of solid slab concrete construction, supported by reinforced concrete columns. An unusual feature of the design calls for cantilevered floor construction at some points. The floor and deck of the two-car garage are also of concrete, the latter so designed as to be convertible into an additional bedroom at a very low cost. Wall construction is of a special type, faced on the outside with portland cement stucco over metal lath; on the inside with Rocklath over which there is a thin coat of plaster. Insulation is provided by inserting

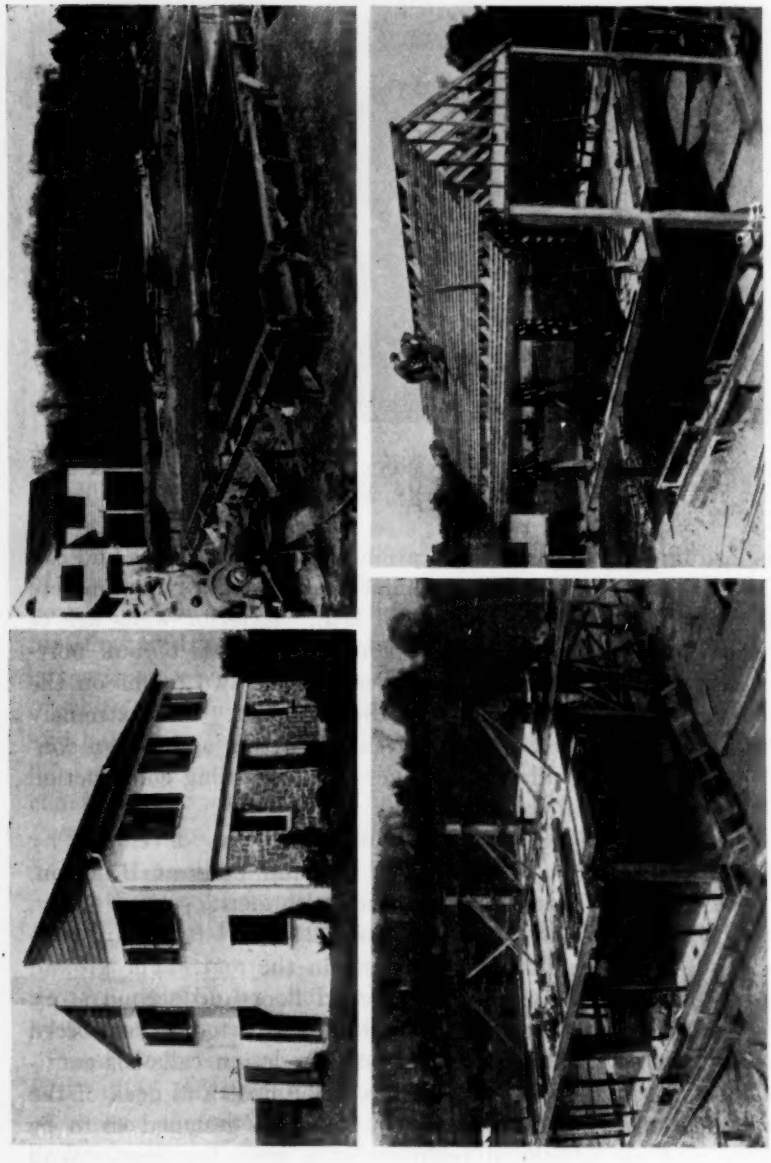


FIG. 7, 10—RESIDENCE BY INTER INDUSTRIES, INC., AT BRIGHTON, N. Y.
C. STORRS BARROWS, ARCHITECT

FIG. 11—RESIDENCE FOR DANIEL S. ROBERTS, WOLLASTON, MASS.
C. M. WILLIS, ARCHITECT

mineral wool in waterproof bags between the vertical wall members. The most interesting feature of this type is its cost—$4,950, including a desirable lot 75 by 120 feet. This price includes gas, water, electricity and other necessary utilities brought into the house; also an approved air-conditioning plant and oil-burner furnace. Financing is arranged by the payment of $1,000 down and carrying and amortizing charges of $37.50 per month. At the end of 15 years, the property passes to the owner.

The Daniel S. Roberts residence at Wollaston, Mass., C. M. Willis, Architect, Lexington, Mass., must also be classified in the low-cost group. The design is of the true Cape Cod type, the walls consisting of 8-in. coursed concrete masonry units, painted with portland cement paint. There is a reinforced concrete floor over the garage, but wood floors have been provided elsewhere. This seven-room and garage home, 44 x 33 ft., was built at a cost of $6,500.

Among the houses in the medium price classification there should be included the Byron Dexter house at Croton-on-Hudson, New York, the work of Harvey Stevenson and Eastman Studds, New York City architects. It is of modernistic design, has exterior walls of 8-in. cinder concrete masonry units, lightly coated with one coat of cream-colored portland cement stucco. This residence which has ten rooms and two baths cost approximately $16.000.

Hopkins & Dentz designed an attractive residence for L. M. Mayer at Scarsdale, N. Y., the approximate cost of which is said to have been $14,000. The exterior walls consist of 8-in. sand-concrete masonry units, an intervening air space of two inches, and an inside 4-in. cinder masonry wall. These units are of colored random ashlar

FIG. 12—L. M. MAYER RESIDENCE, SCARSDALE, N. Y.
HOPKINS AND DENTZ, ARCHITECTS

pattern. There is a concrete floor over a portion of the basement; otherwise the floors are of wood.

Another house which should surely be included in this classification is that just completed by Kings Builders, Inc. at Teaneck, New Jersey. Wall construction consists of 4-in. sand-concrete masonry units on the exterior, colored by proper selection of aggregates, an air space of 1½ in. and a 4-in. cinder concrete masonry unit on the interior, making a wall thickness of 9½ in. Interior partitions are either of 4- or 8-in. cinder concrete units. In some cases the units

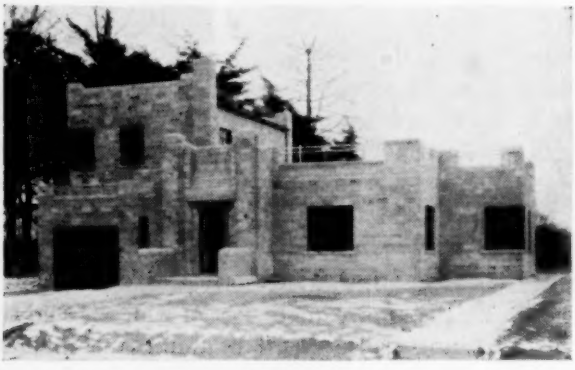


FIG. 13—RESIDENCE BY KINGS BUILDERS, INC., TEANECK, N. J.

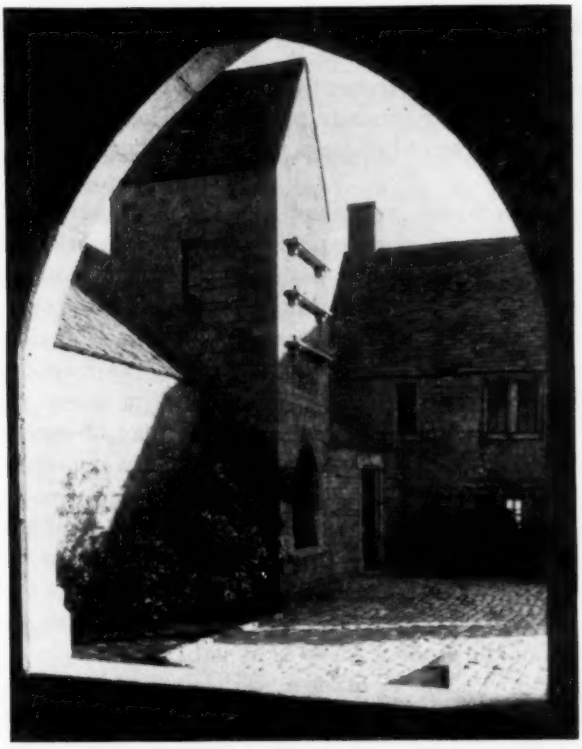


FIG. 14—CLOSE-UP OF W. A. FLINN FARM GROUP, BANKSVILLE, CONN. ALFRED HOPKINS AND ASSOCIATES, ARCHITECTS

have been left exposed, as manufactured; in others, paint of the desired color has been applied, but no plaster has been used anywhere in this house except on some suspended ceilings. Floors are of precast cinder concrete joists over which was placed a 2½-in. gravel concrete slab. In this thoroughly fireproof home the living room is segregated from the remainder of the house by being placed on the second floor. In some rooms, the precast joists are left exposed and painted; in other cases, a furred and plastered ceiling has been applied. Opened to the public less than two weeks ago, this house has attracted many thousands of people who have been much interested in its unusual modernistic design.

In the higher cost classification, only one residence group will be described, that designed by Alfred Hopkins Associates, New York City architects who were pioneers in seeing and taking full advantage

of the unexcelled features of concrete masonry units in their many forms, textures and colors. The W. A. Flinn farm group near Banksville, Connecticut, is an excellent example of the use of textured sand-concrete coursed ashlar concrete masonry. The group is a large one, consisting of a number of buildings which constitute the farm-residence of Mr. Flinn.

The architects whose work has been briefly described, as well as many others who have done equally meritorious work, have realized that the modern home can be designed to retain all of the excellent advantages of older types; obtaining in addition, however, fireproofness, permanence, strength and beauty—for there is available a much wider variety of form, color and texture, where concrete is used.

Not only has concrete been employed in masonry units in the northeastern area, but also in a number of types of precast sections. Several architects or builders have recently developed types which vary widely in conception and application. Prominent among such systems is one developed by L. M. Parkhurst of Lackawanna, New York. In this case the exterior wall consists of 4- or 6-in. precast concrete I-shaped studs, depending upon the wall loads to be carried. Precast concrete slabs of haydite concrete, ¾ x 10 x 34 in., are attached to either side of the studs. These thin slabs may be plain, colored or surfaced with brick slabs or other materials. The studs are reinforced with three ½-in. bars, and the slabs with wire mesh of some type. Footings consist of reinforced concrete precast slabs 12-in. wide, laid on a bed of cinders. Joints between the individual slabs are effected with metal strips which key into each other and set in a specially selected mastic material. Steel ties, which project from the studs are grouted to the thin slabs. Floor and roof construction is on channel-shaped precast concrete slabs. For the roof these slabs are cast to simulate shingles, tile or clapboards.

Definite progress has been made by Parkhurst Systems, Inc. toward approval for the use of their form of construction. It is reported that the Subsistence Homesteads Division at Washington has tentatively approved the system for Subsistence Homesteads projects. Several houses employing the system are under construction in Lackawanna. Mr. Parkhurst has made the statement that in 100-home groups, one-story houses can be built at a cost of $1,608 or 16¢ per cu. ft., and two-story houses for $1,800. These prices are for a complete house, including electric wiring and a hot air furnace installation.

Arthur H. Olmsted formerly of Rye, New York, has developed a system of wall construction which has several points of interest. Over the usual foundation wall there is placed one of the many avail-

able types of poured-in-place concrete floor slabs. Over the slab are set precast concrete studs of a special design and 2 inches by 7 inches in section. A lath, containing projecting nail heads to key into the concrete, is incorporated on one or both faces of the studs. Ordinary wood panel forms are used on the exterior face of the wall, in order to retain the concrete which is placed between the studs. On the inside face a confining surface is formed by bending some type of insulating board between the studs. An air space results, as shown in the diagram, the insulation board remaining in place. Space for pipes, wires, etc. is also thus provided. The studs are vertically reinforced and horizontal bars may easily be placed in the stud voids as concreting proceeds. Another reinforced concrete floor slab is placed over the first-story walls; other precast studs are set upon the slab and the second story proceeds in the same manner as the first.

The outside surface of the wall is easily stuccoed, veneered with brick or some type of concrete masonry unit, or covered with clapboards or shingles as desired. Careful figures made on the basis of a number of houses already built in Westchester County, New York indicate the cost of houses so constructed to be not in excess of six or seven per cent above that for wood construction.

Still another type of precast concrete system has been developed by Quentin Twachtman, Greenwich, Conn., architect. This consists essentially of wall units 9 ft. 6 in. x 15 ft., cast in one operation. A one-in. layer of Pottsite concrete, in which there is embedded a 6 x 6 wire mesh, is first laid on the wooden form. Then crimped metal lath, forming ribs at intervals of 16 in. on centers is placed into which and over all of which another inch of concrete is deposited. Vertical reinforcement is placed in the ribs. This makes a wall section 8. in thick. After properly drying and curing, the large units are transported to the building site on a specially constructed truck and erected in place in a short time. These precast sections are used both for walls and floor or roof slabs. Such a house has been erected at Riversville, Conn., consisting of five rooms and two baths, a maid's room and bath, a furnace room and a one-car garage. Walls are not plastered but simply painted on the interior. Exterior finish may be of stucco, brick veneer or other similar material. According to Mr. Twachtman the cost of such a thoroughly fireproof house is no more than one built of wood.

There is a third development, localized largely in the metropolitan New York area, which is concerned with low-cost, but, nevertheless, large scale housing which is of particular interest at this time. For many years it was thought that housing for larger numbers could be

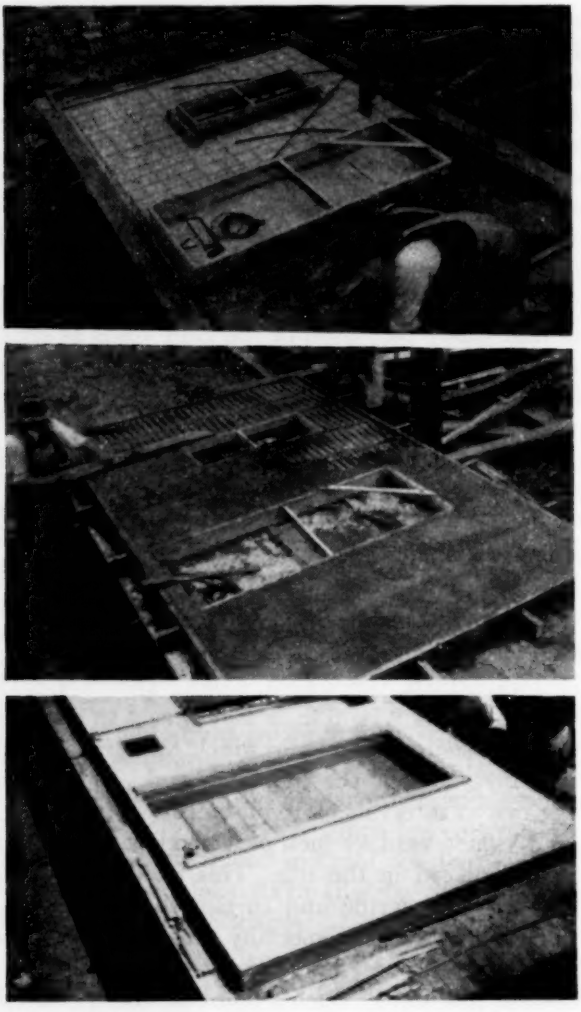


FIG. 15, 16, 17—MANUFACTURE OF PRECAST WALL UNIT—TWACHTMAN SYSTEM

done only with a structural steel frame, when combined with the typical New York "cinder arch" floors. In 1928 extensive surveys were conducted by the writer, with the result that it was definitely shown that reinforced concrete structural frames, combined with any one of several types of long span stone or gravel concrete floors, could be economically designed and built. Using the writer's surveys as a basis, a number of the leading New York architects, engineers and

FIG. 18—CONSTRUCTION OF FLAGG COURT APARTMENTS, BROOKLYN, ERNEST FLAGG, ARCHITECT

builders have proved conclusively in actual construction that reinforced concrete is speedy of construction, while at the same time somewhat more economical than other types previously used. The use of reinforced concrete structural frames and long span concrete floors will be described on two large and recent projects.

The Flagg Court Apartments on Ridge boulevard in the Bay Ridge section of Brooklyn were designed by Ernest Flagg, well-known New York architect, and built by White Construction Co., Inc., New York City, in 1932. Four units of the projected group of eight 9-story and basement apartments were built at that time. Each unit covers a plot approximately 100 ft. square and except for connections with other units at the roof and basement, is complete in itself.

The structural design of this project, prepared by R. L. Bertin, Chief Engineer of White Construction Company, Inc., was based on the elastic theory, without reference to the usual conventional moment coefficients. A reinforced concrete frame of unusually slender proportions was developed with thin slab floors reinforced in two directions. Columns were so located as to occur only in closets, and flush with partitions on the room sides. Beams project into the rooms only at partitions, which are of two inches of cement plaster, placed over wire

mesh. Five ply Douglas fir plywood, ½ in. thick, which proved sufficiently strong to carry the weight of five inches of concrete without undue deflection, was used for the slab forms. The ceilings so developed were so smooth that paint only was applied as the finished surface. The ten stories were built with form panels for only one complete floor plus the equivalent of half a floor for replacements. The design of the structural concrete was based on a 28-day strength of 2500 p.s.i. for the floors and 3000 p.s.i. for the columns. The significant part of this project was the fact that comparative estimates which were made by the designers convincingly proved that the cost of the entire structure was below that of any other type of fireproof construction which might have been selected. These estimates took due consideration of the savings effected by reductions in total wall heights, due to the elimination of ceiling plaster, floor fill and finish.

Another large project on which similar economy and advantages were conclusively shown was a group of six apartment buildings, each 26 ft. 6 in. wide, four of which are 5 stories high and the others six stories. These were built in 1933 for the Prudential Insurance Co. in Newark, by the Walter Kidde Constructors, New York City contractors, in two groups, each approximately 640 ft. long.

The structural frames throughout are of reinforced concrete, as are several full height dividing walls in each unit. Floors are of solid slab concrete and ceilings have not been plastered, but painted to provide the desired finish.

Plans originally prepared by the architects for the Prudential Life Insurance Co. called for structural steel frames with cinder concrete floor arches. The architects also prepared an alternate scheme calling for the use of gypsum lumber floor construction in combination with structural steel framing. However, at the instance of Voorhees, Gmelin & Walker, consulting architects on the project, another alternate calling for reinforced concrete structural frames and floors was prepared. It was the latter alternate on the basis of which the contractors submitted the lowest bids, resulting in a material saving to the owners, and cubic foot costs of somewhat less than 40 cents.

Another similar group for the same owners is now under construction by the E. C. Waldron Co., general contractors of Newark, N. J. When bids were submitted, it was found that, without exception each contractor's bid was considerably lower for the reinforced concrete structural frames and floors than for the steel framing when used in combination with other types of floor construction.

The two cases which have been described somewhat in detail, point the way very definitely to economies of construction never before

realized not only for large-scale, low-cost housing, but also for all light live load structures, including apartments, schools and institutional buildings of various types. Heretofore but little consideration has been given to the use of reinforced concrete for the structural frames and floors of such buildings. Other projects which are now being planned will still further demonstrate the several advantages which these two projects have conclusively shown.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by Aug. 1, 1935.*

Structural and Economic Studies of Monolithic Concrete Walls for Dwellings*

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In this paper the authors propose to summarize some recent studies pertaining to the design, construction and cost of monolithic concrete walls for dwellings. Considerable ground will be covered; therefore it will be necessary to deal somewhat briefly with each study.

The monolithic concrete house is a live issue. Interest in it continues in almost every section of the country. Each year a number of monolithic houses are built and in addition hundreds of people inquire for information regarding the design and construction of this type of house. It is a steady and increasing interest and seems to be more deeply rooted today in the demand for more permanent and better built dwellings than ever before.

The kind of information which builders and architects in particular desire is extremely varied but for the most part regards some phase of structural design, form work and thermal insulation. The following discussion will deal with some of the phases of these subjects.

CONSIDERATIONS IN THE DESIGN OF MONOLITHIC DWELLING WALLS

The design and construction of monolithic dwelling walls involve a number of considerations including structural strength, fire resistance, weathertightness, thermal insulation, surface and architectural treatment and the practicability and cost of application.

The structural design or determination of the wall thickness or cross-section, reinforcement and other related details affect the cost as well as the strength or safety of the structure.

BUILDING CODE REQUIREMENTS

Few building codes specifically cover reinforced monolithic walls for dwellings. Examination of the codes of 15 large cities disclosed

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only four which did so and these imposed a minimum thickness of 5.66 to 8 in. for exterior bearing walls and a reinforcement ratio ranging from 0.1 to 0.3 per cent. It was not always clear whether the reinforcement percentage refers to the total amount or is required in each direction, horizontally and vertically.

Plain concrete frequently is classed as unit masonry and the same wall thickness required. An exception is in the "Recommended Minimum Requirements for Small Dwelling Construction" of the Building Code Committee, U. S. Department of Commerce in which a minimum thickness of 6-in. is recommended for solid monolithic bearing walls containing less than 0.2 per cent reinforcement. It does not make any recommendation in regard to walls with a larger amount of reinforcement.

It thus appears that codes in general do not offer much guidance to the architect or builder of a reinforced monolithic concrete house. In most cases he would have to prepare the plans and then submit them to the building department for special approval. This situation probably contributes to a large extent to the wide variations in wall thickness and reinforcement used under the same conditions in actual practice. Obviously, either the thinner walls used are unsafe, or the thicker ones are uneconomical.

WALL LOADS

The stresses in walls are due principally to gravity loads and wind pressure. In some localities earthquake forces also must be considered.

In determining the wall thickness or the cross-sectional area, it is generally sufficient to consider only the stresses resulting from gravity loads. These include the dead and live loads of that part of the structure supported by the wall at the section under consideration. The position and direction of the resultant of the loads may be determined and the stresses calculated.

Table 1 shows the vertical wall loads in a typical 2-story house with light and heavy types of concrete construction, thus indicating the range of loading which may be expected in concrete dwellings.

Fig. 1, based on Table 1, shows the accumulated vertical loads at different points of the exterior wall and load bearing partition. The critical section in exterior walls generally is just above the sill line in the first story where window and door openings reduce the net wall section available for bearing. The critical section in load bearing partitions generally will be either at the first floor or the basement floor lines.

TABLE 1—VERTICAL WALL LOADS IN TWO STORY HOUSE—LIGHT AND HEAVY TYPES OF CONCRETE CONSTRUCTION

Building Element	Description of Light and Heavy Types of Construction	Unit Loads p.s.f.		Wall Load lb. per lin. ft.	Accumulated Load on Exterior Walls, lb. per lin. ft.		Accumulated Load on Load Bearing Partition, lb. per sq. ft.	
		Dead Load	Live Load		Light Construction	Heavy Construction	Light[1] Construction	Heavy[2] Construction
Roof	2x6's Wood sheathing, Cement asbestos shingles	10	20	540	540	1280		
	Precast Joists; 2 in. conc. slab, tile roofing insulation	49	20	1280				
Attic Floor	Precast joists, 2 in. conc. slab, plaster ceiling	39	40	630	1170	2265	1260	1970
	6-in. solid slab, ½-in. topping, plaster ceiling	83	40	985				
Walls 2nd Story	6-in. Ribbed conc. wall, insul. & plaster	53		470	1640	2965	1810	2645
	2-in. & 4-in. conc. walls with insul. between	78		700				
2nd Floor	Precast joists, 2-in. conc. slab, wood finish floor, plaster ceiling	43	40	665	2305	3940	3140	4595
	6-in. solid slab, wood finish floor, plaster ceiling	82	40	975				
Walls 1st Story	6-in. Ribbed conc. walls with insul. and plaster	53		470	2775	4640	3690	5270
	2-in. & 4-in. conc. walls with insul. between	78		700				
1st Floor	Precast joists, 2-in. slab, wood finish flooring	35	40	600	3375	5585	4890	7160
	6-in. solid slab, wood finish flooring	78	40	945				
Basement Wall	8-in. monolithic concrete	100		850	4225	6435	5530	8010
Footing	8-in. x 16-in.			130	4355	6565	5660	8250

[1]4-in. solid in 1st and 2nd stories, 6-in. solid in basement.

[2]6-in. solid in 1st and 2nd stories, 8-in. solid in basement; 12-in. by 24-in. footing.

The vertical load at the first story sill line of the exterior wall is 2580 lb. and 4370 lb. per lineal foot of wall respectively, for the light and heavy types. If the openings amount to 35 per cent of the horizontal sectional area, the load per lineal foot must be increased by 54 per cent, giving loads of 3970 and 6740 lb. per lineal foot, or the

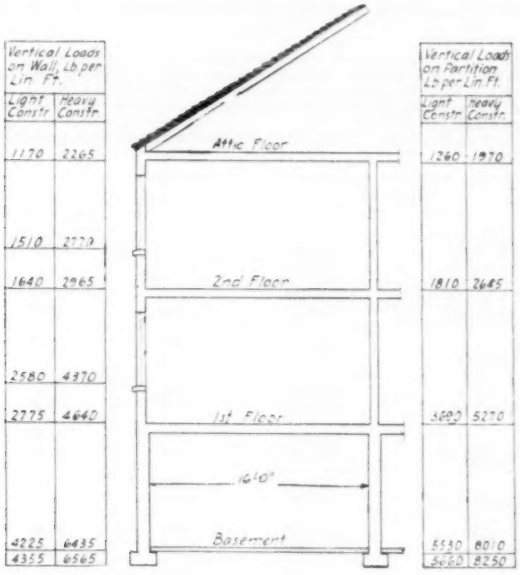


FIG. 1—TYPICAL SECTION THROUGH DWELLING SHOWING VERTICAL WALL LOADS AT DIFFERENT POINTS

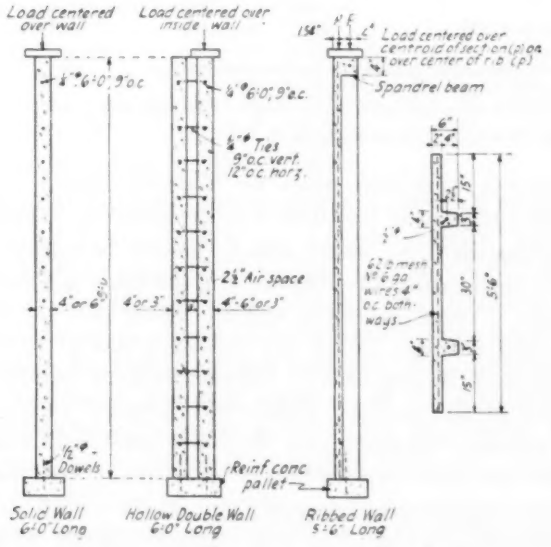

Load centered over wall
Load centered over inside wall
Load centered over centroid of sect. on (plus over center of r.b. (p.)
Spandrel beam
4"ø 6'0" 9" o.c.
4"ø 6'0" 9" o.c.
4"ø Ties 9" o.c. vert. 12" o.c. horiz.
2½" Air space
4" or 6"
4" or 3"
4"-6" or 3"
6x2 mesh No. 6 ga. wires 4" o.c. both ways
½"ø Dowels
Reinf. conc. pallet
Solid Wall 6'0" Long
Hollow Double Wall 6'0" Long
Ribbed Wall 5'6" Long

FIG. 2—DETAILS OF TEST WALLS

TABLE 2—PRINCIPAL RESULTS OF COMPRESSION TESTS OF SOLID, DOUBLE AND RIBBED WALLS

All walls 9 ft. high by 6 ft. long except walls in Group 8 which were 9 ft. by 5 ft. 6 in.
Walls in Groups 1 to 7 inclusive and in Group 9 made of dry tamped concrete placed in 9 in. courses. Walls in groups 8 and 10 made of plastic concrete, 5-8 in. slump.
Axial loading used for all walls except 8A and 8B where the eccentricity = 2.46 in. For double walls, the load was applied axially to the inside wall.
Walls and control cylinders cured in air of laboratory, tested at 28 days. Cylinder strengths are average of 6 to 12 tests.

Group and Wall No.	Description of Wall	Ultimate Strength of Wall		Ave. Control Cyl. Strength p.s.i.	Ratio of Wall Strength to Ave. Cyl. Strength Per Cent
		Lb. per lin. ft.	p.s.i.		
1A	Solid 4-in. wall	99,400	2070	3600	57.4
1B		143,000	2980	3050	97.7
2A	Solid 4-in. wall	152,500	3180	5790	55.0
2B		210,500	4380	5850	74.9
3A	Solid 6-in. wall	235,000	3260	3280	99.4
3B		229,000	3180	3070	103.5
4A	Double 4-in. wall	126,800	2640	3120	84.5
4B		138,500	2880	3480	82.7
5A	Double 3-in. wall	68,750	1910	3480	54.9
5B		96,900	2690	4340	62.1
6A	Double wall 6-in. and 4-in.	227,500	3160	3760	84.0
6B		213,000	2960	3790	78.2
7A	Double wall 4-in. and 3-in.	154,600	3220	3810	84.6
7B		89,800[1]	1870[1]	3790	49.2[1]
8A	Ribbed wall	16,150	3620[2]	2610	18.6[3]
8B		11,500	2700[2]	3460	10.3[3]
8C		58,000	1900[2]	3130	57.2[3]
8D		53,450	1750[2]	2720	59.6[3]
9A	Solid 4-in. wall	73,500	1530	2120	72.2
9B		77,300	1610	1730	93.2
10A	Solid 4-in. wall	105,500	2200	3190	69.0
10B		115,000	2400	2910	82.3

[1]Failed in course damaged by tamping during construction.
[2]Stresses computed assuming straight line distribution, n = 10.
[3]Equals $\frac{P}{F}$ where P = ultimate load and $F = A_c f'_c + A_s f_s$ (yield point)
A_c = 159.1 sq. in. $A_s f_s$(yield point) = 60,500 lb.

reduction in net wall area must be otherwise taken into account. The loads on the bearing partition may be similarly analyzed.

Special problems arise where closely spaced windows may result in relatively high load concentrations in the narrow piers and at the outside jambs. These generally will be within safe limits where the wall is of solid section. With the lighter types of ribbed walls, it may be necessary to use heavier rib members or additional vertical reinforcement at the jambs. Reinforced spandrel areas of sufficient depth and rigidity will distribute the loads uniformly along the wall and minimize the increase in stress at the edges of openings.

Wind loads usually are assumed from 15 lb. to 25 lb. per sq. ft. uniformly distributed over the wall surface. Their principal effect in the case of monolithic dwellings is to produce bending moment in

TABLE 3—PRINCIPAL RESULTS OF FLEXURE TESTS OF SOLID, DOUBLE AND RIBBED WALLS

Single concentrated load applied at center of 8 ft. span.
Walls 1T, 4T and 9T built in 9 in. courses of dry tamped concrete walls. 10T, 8T, 8BB and 8AA built of plastic concrete of 5-8 in. slump.
Walls and cylinders cured in air of laboratory, tested at 27 or 28 days except walls 8BB and 8AA which were tested at 38 and 45 days respectively.
Cylinder strengths are average of 6 to 10 tests.

Group and Wall No.	Description of Wall	Manner of Failure	Point of Failure[1]	At Section Where Failure Occurred			Equivalent Wind Pressure, Lb. per Sq. Ft.[2]	Average Control Cylinder Strength, p.s.i.
				Moment, per Foot of Wall In. Lb.	Flex. Stress p.s.i.	Resultant Modulus of Rup-p.s.i.[3]		
1T	Single 4 in. wall	Tension in concrete	Top of course 5	2,240	70	63	18	3350
4T	Double 4 in. wall	Tension in concrete	Top of course 5	4,370[4]	137[4]	130[4]	72	4340
9T	Single 4 in. wall	Tension in concrete	Top of course 4	2,150	67	59	18	2120
10T	Single 4 in. wall	Tension in concrete	Center	13,200	413	407	108	3160
8T	Ribbed wall	Tension in steel	Center	30,000			246	2720
8BB	Ribbed wall. Retest of 8B in flexure	Tension in steel	Center	23,600			194	2610[6]
8AA	Ribbed wall. Retest of 8A in flexure	Compression in rib		35,600[5]			292	3460[6]

[1]In walls 1T, 4T and 9T of dry tamped concrete, top of course 6 is center of wall.
[2]Flexural stress with compressive stress of 6 to 8 p.s.i. deducted.
[3]Computed from moment values corrected to apply to 9 ft. span.
[4]Each part of double wall assumed to take one-half of thrust.
[5]Thrust applied as from inside of wall, ribs in compression.
[6]Strengths at 28 days.

the vertical wall span between floor levels. Ordinarily the overturning moment, shearing stresses, and other effects of wind pressure may be disregarded. Maximum stability will depend greatly on how well the different structural parts are tied together.

STRENGTH TESTS OF MONOLITHIC CONCRETE WALLS

Tables 2 and 3 show data on the compressive and transverse strengths of different types of monolithic walls developed in an investigation recently completed by the Engineering Experiment Station, University of Illinois in cooperation with the Portland Cement Association. Frank E. Richart, Research Professor of Engineering Materials and N. M. Newmark, Research Assistant in Civil Engineering, were in charge of the tests. The complete report by Messrs. Richart and Newmark is not yet available in published form.

Description of test walls

Fig. 2 shows the details of the different types of walls tested. These included ribbed walls having a 2-in. slab and light ribs 33-in. on center;

single solid walls 4-in. and 6-in. thick and double walls composed of two 4-in. walls, two 3-in. walls, 3-in. and 4-in. walls, and 4-in. and 6-in. walls. The air space was 2½-in. for the different double walls. There were 27 large walls, 9 ft. high by 5 ft. 6 in. or 6 ft. long, and 6 small walls, 4 ft. high by 32-in. wide. Compression tests were made on 22 of the large walls; 5 walls were tested in flexure. Two of the large walls of the ribbed type tested in compression were later retested in flexure, making a total of 7 flexure tests. Results of the compression tests on the small walls are omitted herein.

Two types of concrete were used. Walls in Groups 1 to 7 inclusive and in Group 9 were made of a dry tamped concrete of no slump. A plastic concrete was used for walls in Groups 8 and 10, the slump being 8 in. and 5-6-in. respectively. Three different dry tamped mixes designed to give strengths of about 2,000, 3,000 and 4,000 p.s.i. were used. Due to the characteristics of this type of mix and the variables in placing, the cylinder strengths showed much greater variation than is common with plastic mixes. The average of the cylinder strengths for each wall tested is indicated in the tables.

The walls of the dry tamp mix were constructed in successive courses 9-in. high, tamped in two 4½-in. layers with a tamper weighing about 5 lb. Successive courses were built at intervals of about 2 hours excepting that 14 to 16 hours elapsed at the daily construction joint.

The side forms consisted essentially of two 12-in. steel channels bolted to wood guide posts set at the ends of the wall. For the double walls, two such guide posts were set at each end, the steel channels were used for the outside forms and 1-in. boards held apart by wedges comprised the inside, or air-space forms.

Construction of the dry tamped walls followed field practice with horizontally sliding molds excepting that the forms were moved up instead of forward. The forms were loosened shortly after filling and reset in position for the next course. The early stability of the dry tamped concrete was sufficient to permit this procedure. Ordinarily, four 9-in. courses were completed in one day.

It should be noted that none of the solid or double walls was reinforced vertically thus conforming with past practice with this type of construction because the presence of vertical reinforcement at any point except at wall corners, interferes with the operation of the sliding molds. Horizontal reinforcement consisted of one-quarter inch round bars 9-in. o.c. in each wall.

Walls in Groups 8 and 10 were cast as in one unit, the forms being filled to the full height of the wall in one day's time. Forms consisted

FIG. 3—SOLID WALL IN TESTING MACHINE

of wood sheathing, studs and wales, spaced and tied together with a spreader-tie device furnished by the Arro-Tie Company.

All walls and control cylinders reported herein were cured in the air of the laboratory. Tests were made at 28 days excepting that walls 8BB and 8AA were tested at 38 and 45 days respectively.

Test methods

Compressive tests were conducted in the 3,000,000 lb. Southwark-Emery hydraulic testing machine. Axial loading was applied through the spherical bearing block and two 24-in. I-beams to the 2-in. thick steel capping plate which was bedded several days before testing in a layer of neat, high early strength cement and left in place. Eccentric loading was similarly applied through a ½-in. cold rolled square bar to the capping plate.

Fig. 3 shows a wall in the testing machine. The vertical deformations and the lateral deflections of the walls were observed at increments of load by means of four attached compressometers and a deflectometer.

In the flexural tests the thrust was applied uniformly through a horizontal steel beam at the center of a span measuring 8 ft. vertically. Steel beams placed laterally provided support at the top and bottom of the wall. The lever device used for applying the load to the distributing beam is shown by Fig. 4. Deflections at the center of the span were measured at increments of load.

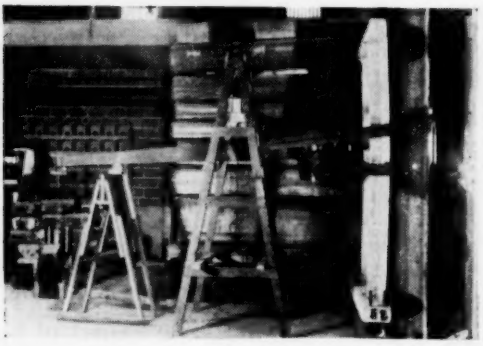


FIG. 4—SET-UP FOR TRANSVERSE TEST

Results of compression tests

The principal results of the compression tests are given in Table 2. All walls except the ribbed walls 8A and 8B were loaded axially. The double walls were loaded only on the inside wall portion. Ultimate loads are indicated in pounds per lineal foot of wall as well as in pounds per square inch. Comparison of the latter values with the cylinder strengths gives the ratios of wall strength to cylinder strength.

The 4-in. solid walls in Groups 1, 2 and 9 were made of different mixes of dry tamped concrete. Ultimate strengths ranged from 73,500 lb. to 210,000 lb. per lineal foot and the ratios of wall strength to cylinder strength ranged from 55.0 per cent to 97.7 per cent averaging 75 per cent. These ratios apparently were not consistently affected by the strength of the concrete. The 4-in. solid walls 10A and 10B made of plastic concrete have a comparable average ratio of wall strength to cylinder strength of 75.6 per cent.

The 6-in. solid walls 3A and 3B gave strengths of 235,000 and 229,000 lb. per lineal foot. The average unit wall stress was greater than the cylinder strength indicating there was no effect from bending.

The double walls developed strengths ranging from 68,750 to 227,500 lb. per lineal foot, varying greatly with the wall thickness and unit concrete strength. Ratios of wall strength to cylinder strength ranged from 49.2 per cent (for wall damaged in construction) to 84.5 per cent, averaging 72.5 per cent. The metal ties connecting the loaded and unloaded portions of the double walls apparently did not serve to increase load capacity.

Ribbed walls 8A and 8B were loaded at the center of the rib with an eccentricity of 2.46-in. Failure occurred in compression in the ribs just below the spandrel beam. The ultimate loads were 16,150 and 11,500 lb. per lineal foot. Ribbed walls 8C and 8D were loaded

axially and developed strengths of 58,000 and 53,450 lb. per lineal foot respectively. These results indicate the importance of designing floor connections so that the stresses will be more uniformly distributed over the wall section. The strength of this type of wall with eccentric loading may be increased substantially by using heavier ribs more closely spaced.

Results of flexural tests

The principal results of the flexural tests are shown in Table 3. All of the tamped walls indicated as 1T, 4T and 9T failed at a daily construction joint, indicating the flexural strength was determined by strength of bond between courses. Modulus of rupture ranged between 59 to 130 p.s.i., the higher value being obtained with the double wall. The values of moment, flexural stress and modulus of rupture for the double wall apply to only one wall portion, it being assumed that one-half the thrust was taken by each portion. The 4-in. solid wall, 10T, made of plastic concrete without a daily construction joint was much stronger in flexure than the tamped walls and developed a modulus of rupture of 407 p.s.i.

The ribbed walls were reinforced vertically and therefore performed differently from the other walls. The flexural strength was dependent on the tensile strength of the steel and the compressive strength of the concrete rather than on the modulus of rupture of the concrete or bond strength between courses as in the case of walls without vertical reinforcement.

The values of equivalent wind pressure are based on the test results and indicate the uniform pressure which would cause failure in each respective type of wall 9 ft. high and supported at the top and bottom as at floor levels and having a vertical load on top of 250 to 315 lb. per lineal ft.

It should be mentioned that while the main purpose of flexural tests is to determine the resistance to failure due to wind pressure, it is impracticable to duplicate exactly actual service conditions. The test specimens were given horizontal support at the top and bottom whereas walls in a dwelling are in addition supported vertically by wall corners, offsets and interior partitions. For walls with no openings, the total pressure is carried in each direction, vertically and horizontally. Where the wall areas are broken by windows and doors, the effect of horizontal resistance is largely nullified except where the horizontal spans are not appreciably greater than the vertical spans.

Another factor is the type of wall section. For instance, the flexural strength of ribbed walls is greater in the direction of the ribs

than across them. Dry tamped walls are stronger in the horizontal span due to the horizontal reinforcement and the absence of vertical construction joints.

Fortunately, an exact analysis of the flexural stresses is unnecessary for most types of monolithic concrete walls which meet vertical load requirements. Such walls generally provide adequate lateral strength with moment carried only on the vertical span. The only exception in so far as the walls tested are concerned, is the 4-in. solid wall of tamped concrete which for practical considerations is not a type proposed for use in actual construction. Where made with plastic concrete the flexural strength of 4-in. solid walls generally will be adequate.

The results of the investigation which bear particularly on this discussion may be summarized as follows:

1. The compressive strength of all axially loaded walls was over 55 per cent of the strength of the concrete control cylinders and averaged about 78 per cent of the cylinder strength.
2. The eccentrically loaded ribbed walls were only 20 to 30 per cent as strong as the axially loaded ribbed walls.
3. Flexural strength of the dry tamped walls was dependent on the strength of bond between courses and was least where the courses were constructed 12 hours or more apart. Values of modulus of rupture varied from 59 to 130 p.s.i. for these walls.
4. Flexural strengths of the solid, poured and ribbed walls are not essentially different from beam strengths of beams of similar section when all elements involved are evaluated.
5. Assuming failure in flexure to take place at the maximum moments recorded in the test walls and further assuming construction joints in the dry tamped walls at mid-height the wind loads that could be carried by the walls on a 9 ft. span are as follows: Solid 4-in. walls of dry tamped concrete—18 lb. per sq. ft.; double 4-in. walls of dry tamped concrete—72 lb. per sq. ft.; 4-in. solid poured walls—108 lb. per sq. ft.; ribbed walls—about 200 lb. per sq. ft.

SAFE WORKING LOADS; COMPARISON WITH DESIGN LOADS

In determining the working loads consideration has been given to the test results and the selection of a reasonable and adequate factor of safety. Comparison of test data on the ultimate compressive strengths of unit masonry walls with the allowable working stresses indicates that this type of wall develops safety factors ranging upward from four. There is considerable precedence for values of 3, 4, and 5 and it appears reasonable to suggest the following factors of safety for story height monolithic concrete dwelling walls:

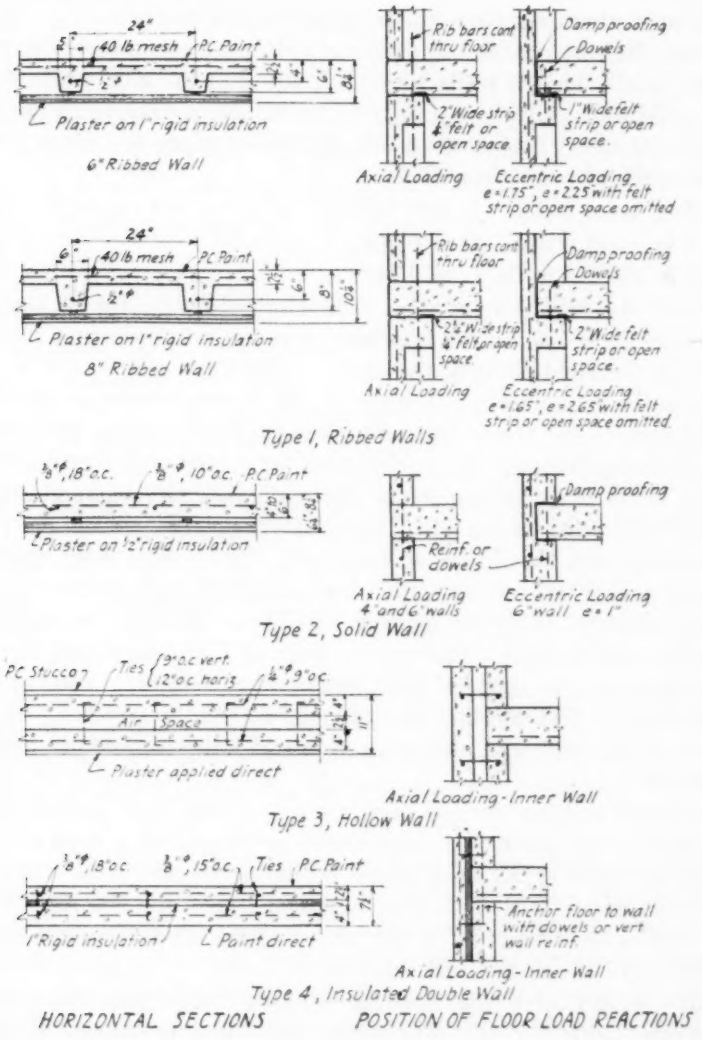

24"
40 lb mesh
P.C. Paint
Plaster on 1" rigid insulation
6" Ribbed Wall
Rib bars cont thru floor
2" Wide strip ½" felt or open space
Axial Loading
Damp proofing
Dowels
1" Wide felt strip or open space.
Eccentric Loading e=1.75", e=2.25 with felt strip or open space omitted
8" Ribbed Wall
2½" Wide strip
2" Wide felt strip or open space.
e=1.65", e=2.65 with felt strip or open space omitted.
Type 1, Ribbed Walls
⅜" ø, 18" o.c.
½" ø, 10" o.c.
Plaster on ½" rigid insulation
Reinf. or dowels
Axial Loading 4" and 6" walls
Eccentric Loading 6" wall e=1"
Type 2, Solid Wall
P.C. Stucco
Ties 9" o.c. vert. 12" o.c. horiz.
¼" ø, 9" o.c.
Air Space
Plaster applied direct
Axial Loading - Inner Wall
Type 3, Hollow Wall
⅜" ø, 18" o.c.
½" ø, 15" o.c.
Ties
1" Rigid insulation
Paint direct
Anchor floor to wall with dowels or vert. wall reinf.
Axial Loading - Inner Wall
Type 4, Insulated Double Wall
HORIZONTAL SECTIONS
POSITION OF FLOOR LOAD REACTIONS

FIG. 5—DETAILS OF VARIOUS TYPES OF MONOLITHIC DWELLING WALLS

Walls built in courses of dry tamped concrete, F. S. = 5
Walls built integrally of plastic concrete, F. S. = 4

The factor of safety is made higher in the case of course construction with dry tamped concrete because it is less homogeneous and uniform in character than plastic concrete construction.

From the test results the minimum ultimate compressive strengths of the axially loaded walls were as follows: 4-in. solid walls, $0.56f'_c$; 6-in. solid walls, $0.78f'_c$; 6-in. ribbed walls $0.57f'_c$. Applying the aforementioned safety factors the working stresses suggested are as follows:

For walls of plastic concrete:

4-in. solid walls, $f_c = 0.14f'_c$
6-in. solid walls, $f_c = 0.20f'_c$
6-in. ribbed walls, $f_c = 0.14f'_c$

Where the wall is reinforced vertically substitute for f'_c the value $[(1 - p)\ f'_c + pf_s\ Y.P.]$ in which p is the reinforcement ratio and $f_s Y.P.$ is the yield point of the steel.

For solid or double walls made of dry tamped concrete use values 80 per cent of the above and in double walls assume the wall portions to act independently of each other in supporting vertical loads. At the daily construction joints the courses of dry tamped concrete should be bonded with a ½-in. layer of 1:2 mortar.

Bearing walls should be at least 4-in. thick. The 4-in. single dry tamped wall is not recommended for load bearing purposes.

The effect of eccentric loading should be included in calculating the safe working load. The above values of f_c are suggested for working stresses at the extreme compressive fiber assuming a linear stress distribution. The bending moment should not be sufficient to cause tension in the concrete unless there is adequate vertical steel to take the stress.

Eccentric bearing on the inside edge of the wall caused by the deflection of the floor may be avoided by providing a strip of felt or similar material or an open space between the floor and wall along the inside edge. This detail is shown in Fig. 5 for the ribbed walls and should be adopted for the other types when justified by the conditions of the design and loading.

Table 4 indicates the vertical working loads and the ultimate transverse loads for the monolithic wall designs shown in Fig. 5. The vertical working loads are based on the aforementioned assumptions. The ultimate transverse loads are largely based on the information developed in the flexural tests. It should be noted that working transverse loads are given for the ribbed walls. The effect of dead load compressive stresses in increasing the transverse strength is

TABLE 4—DESIGN DATA ON DIFFERENT TYPES OF MONOLITHIC CONCRETE WALLS

Assumptions:
f'_c = 3000 p.s.i.
Concrete takes tension in resisting transverse loads except in Wall Type 1.
Modulus of rupture of concrete = 400 p.s.i. for Wall Types 2 and 4; 130 p.s.i. for Wall Type 3.
In Wall Type 1, steel in rib assumed to take all tension, f_s = 20,000 p.s.i.
Values are for walls with no openings.

Wall Type	Description	Eccentricity of Vertical Load, in.[1]	Safe Vertical Working Loads; Lb. per Lin. Ft. of Wall	Ultimate Transverse Loads; Lb. per Sq. Ft. of Wall	Additional Ultimate Transverse Loads Allowed for Each 100 Lb. per Sq. In. Compression[2]	Coefficient of Heat Transmission "U"[3]
1—6-in. Ribbed Wall	2½-in. slab, 4-in. by 3½-in. ribs 24-in. o.c., 1-in. insulation and plaster, painted exterior	2.25 1.75 0	3,025 3,725 16,600	55[4]	0	0.20
1—8-in. Ribbed Wall	2½-in. slab 5-in. by 5½-in. ribs 24-in. o.c., 1-in. insulation and plaster, painted exterior	2.65 1.65 0	4,700 6,600 19,750	82[4]	0	0.20
2—4-in. Solid Wall	4-in. concrete, with plaster on ½-in. insulation furred out, painted exterior.	0	20,000	106	26	0.28
2—6-in. Solid Wall	6-in. concrete with plaster on ½-in. insulation furred out, painted exterior	1 0	21,500 43,000	236	59	0.27
3—Hollow Wall	2—4-in. walls with 2½-in. air space between. Built in 9-in. courses of dry tamped mix. Exterior stuccoed. Interior plastered direct.	0	16,000	68	52[5]	0.42 0.10[6]
4—Insulated Double Wall	2½-in. concrete, 1" insulation 4-in. concrete. Surfaces painted direct.	0	20,000	130	33[7]	0.24

[1]Refer to Fig. 5.
[2]Refers to average unit compression due to dead load only.
[3]Calculated heat loss in Btu per hour per sq. ft. per deg. F. difference in air temperature with 15 mph wind on outside surface. Values based on data contained in American Society of Heating and Ventilating Engineers Guide.
[4]Values are for working loads.
[5]Value is for each 100 lb. per sq. in. dead load compression in outer wall portion.
[6]Space filled with granulated cork.
[7]Value is for each 100 lb. per sq. in. dead load compression in inner wall portion.

conveniently indicated as additional transverse load for each 100 p.s.i. average unit compression. In the case of eccentrically loaded walls the compressive stress is not uniformly distributed and it would be more exact to use values of extreme fiber stress. However, assuming that the wind pressure is applied on the outside of the wall, the use of average unit compression values would be on the safe side.

The loads indicated in Table 4 for the 8-in. ribbed wall are based on the same working stresses as suggested for the 6-in. ribbed wall. While no tests were made of the 8-in. ribbed wall its ultimate compressive and transverse stresses should be at least as great as for the 6-in. ribbed section.

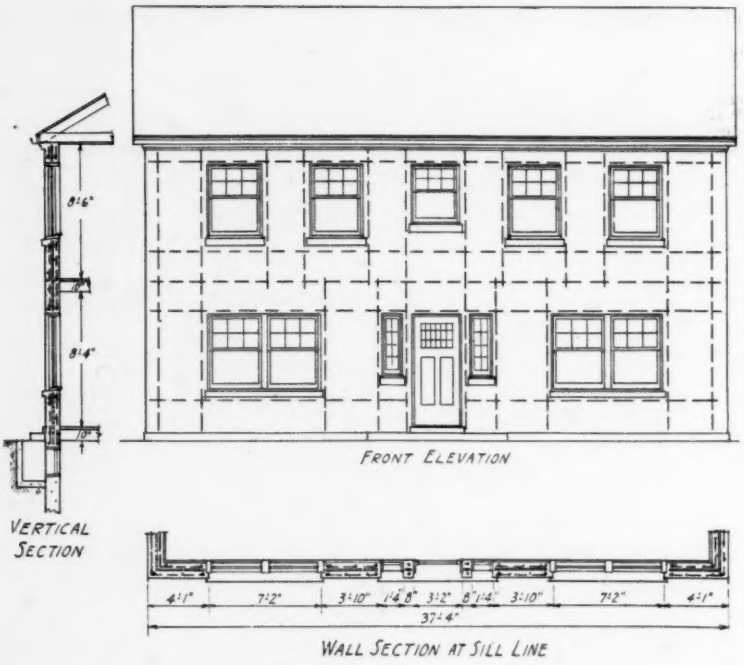


FIG. 6—TYPICAL RESIDENCE SELECTED FOR COMPARISON OF DESIGN LOADS WITH SAFE WORKING LOADS AND TO SHOW LOCATION OF PRINCIPAL REINFORCEMENT

It will be assumed in the following examples that the wind load is carried wholly on the vertical span between floor levels: the horizontal moment will be neglected.

The gravity loads may be compared with the working loads given in Table 4 on the basis of the colonial type dwelling illustrated by Fig. 6 which was selected because of the generous window area.

From Fig. 1, the vertical loads at the first story sill line, which is the critical section, are 1980 and 3470 lb. per lineal foot respectively for light and heavy types of concrete construction.

Considering first the light construction which is of the ribbed type shown in Fig. 5 and described in Table 1 the total vertical design load to be supported along the entire wall length at the sill line is 37.3 x 2580 or 96,200 lb. The net wall length is about 17 ft. The average load therefore is 5,660 lb. per lineal foot of net wall length.

From Table 4 the 6-in. ribbed wall is shown to be adequate provided it is axially loaded. It is inadequate with eccentric loading. To

obtain axial loading the floor construction must be carried through to the outside of the wall in the manner indicated in Fig. 5. Except in mild climates this detail is apt to cause more or less trouble from the direct conductance of cold and moisture. It probably will be more satisfactory to select the 8-in. ribbed wall with a floor bearing detail that reduces the eccentricity to 1.65-in. or less. The working load of 6600 lb. per lin. ft. exceeds the design load.

The load at the second story sill line is 1510 lb. per lineal foot which gives a load of 2810 lb. per lineal foot of net wall section. This is satisfied by the 6-in. ribbed wall provided the eccentricity of the floor and roof bearing does not exceed about 2.25-in.

Assuming a 20 lb. per sq. ft. wind load, the total wind load to be carried by the full length of the wall one-story high is about 6,700 lb. The flexural strength is based on the tensile strength of the steel and the effect of compression may be disregarded as well as the effect of any horizontal moment. It is assumed that the wind load is carried only on the vertical span and by the net wall section at the window line. The total wind pressure may be converted into load per sq. ft. of net wall area by dividing 6,700 by 17x9, giving a load of 44 lb. per sq. ft. Referring again to Table 4, it is seen that this value is less than the working loads for both the 6-in. and 8-in. ribbed walls.

Considering heavy construction of the type indicated as type 4, Fig. 5, the total vertical design load at the sill line of the first story is 4370 x 37.3 or 163,000 lb. The load on the net wall section is 9,580 lb. per lineal foot. This is well within the safe design load of 20,000 lb.

The wind pressure would be the same as for light construction or 44 lb. per sq. ft. on the net area. The ultimate value is 130 lb. per sq. ft., plus the effect of the compressive stress due to dead load.

The dead load compression of about 110 lb. per sq. in. in the inner wall portion will add about 37 lb per sq. ft. to the value of transverse strength. The ultimate transverse load is 167 lb. per sq. ft. or 3.8 times the design load.

The question will arise as to what may be considered a proper factor of safety to use for wind loads. In this connection it may be mentioned that there is considerable variance between actual experience and test results. It is factual knowledge that most types of unit masonry walls have a low flexural strength yet the service record of properly built masonry walls shows comparatively few failures due to wind. It may be that the assumed loads of 15, 20 and 25 lb. per sq. ft. exceed the maximum pressures experienced in most locations. It is also true that several strengthening effects usually are neglected in a theoretical analysis. The effect of the horizontal moment already has been

mentioned. Continuity over supports or end restraint is present in some degree but generally is neglected. Window frames have an appreciable structural value and undoubtedly take some portion of the vertical moment. Of the monolithic walls indicated in Fig. 5 and Table 4, the hollow double wall which has been used for several thousand structures has the lowest flexural strength. Its factor of safety is slightly less than two when analyzed on the basis of a 20 lb. wind load and the design in Fig. 6. No failures of this type of construction due to wind have ever occurred to the authors' knowledge.

In view of these considerations the authors are inclined to feel that a comparatively low factor of safety will be sufficient. The ultimate transverse strength probably should be sufficient to resist at least 1.5 to 2 times the design wind pressure. The factor of safety should be increased somewhat if the various strengthening effects are evaluated and taken into account.

The other types of walls shown by Fig. 5 may be analyzed similarly for the house design in Fig. 6 or any other given design.

Reinforcement

Proper proportioning and distribution of the wall reinforcement is dependent largely on the intensity and distribution of the internal stresses in the concrete caused by volume change. Except in light construction, such as the ribbed wall, stresses resulting from external forces are secondary to shrinkage and temperature stresses.

The required amount of reinforcement frequently is specified as a percentage ratio of the concrete section. This implies uniform distribution over the wall area which generally will prove both unsatisfactory and inefficient where the walls have openings as in dwellings. In such walls, the intensity of the internal concrete stresses varies considerably in different parts of the wall. Maximum stresses usually occur horizontally in areas above and below the openings as a result of the reduced sectional area of the wall.

Examination of monolithic concrete residences will usually show that where cracking has developed, it is most serious in the form of cracks that radiate from the corners of the openings or extend upward from the lintel or downward from the sill. It is logical that additional horizontal reinforcement should be placed in these areas.

For this purpose continuous bars at the sill and lintel lines will be more effective than the short lintel bars commonly used at present. The short bars may adequately resist bending stresses, but unless they are hooked at the ends or extend a distance of 24 to 30 bar diameters on each side of the opening, they will not be as effective as continuous bars.

In determining the distribution of the reinforcement horizontally and vertically, a factor which should be considered is the compression in the wall which tends to resist vertical expansion or the development of horizontal cracks. Cracking most generally extends vertically or diagonally. It appears therefore that the horizontal reinforcement is of more value and should be greater in area than the vertical reinforcement. There should, however, be vertical bars at the edges of all openings, and in narrow piers between openings. Wider areas between windows and doors seldom show cracking and may be reinforced with a minimum amount of steel. Fig. 5 shows the amount of reinforcement considered satisfactory for the different types of monolithic walls shown. Fig. 6 indicates the most effective location of the principal portion of the reinforcement.

Effect of wall design on cost

It appears that monolithic concrete dwelling walls may be safely built of somewhat lighter construction than commonly used. The 4-in. solid section and the 6-in. ribbed wall are structurally adequate for most one story and some two story designs. Where the solid wall is used singly as in Type 2 Fig. 5 practical considerations limit it to one story houses excepting under conditions which permit the floor construction to be extended through to the outside face of the wall without serious consequences from the direct conductance of dampness and cold.

The savings affected by lighter construction are principally in regard to the material and placing cost of the concrete and may amount to 4 to 6 cents per sq. ft. of wall for a 2-in. reduction in average thickness. Other items remain essentially unchanged.

The cost of monolithic walls is composed of formwork, concrete and reinforcing materials, provision for insulation and installation of services, and the exterior and interior wall treatments. Each item should be carefully considered with a view to reducing its cost without detriment to the resulting wall.

Referring to Fig. 5, the two most economical wall types shown are the 4-in. solid wall and the insulated double wall. The first obtains its economy from the savings in concrete materials, simplicity of formwork and the inexpensive exterior treatment. The insulated double wall also uses solid wall forms and while the insulation item will be higher there is a considerable saving effected by eliminating the furring and plaster on the inside.

The ribbed wall requires the least amount of concrete but the form costs will be somewhat higher than for solid walls. The chief economy of the hollow wall is derived from the built-in air space insulation

which to a certain degree takes the place of the furring and rigid insulation shown for the other types. If the air space is filled with granulated cork or equally good insulating material the hollow wall has an excellent insulation value.

In the past too much expense has gone into the structural part and too little into insulation. If a 4-in. wall will support the loads within safe working stresses there is no advantage in a 6-in. or 8-in. wall. The difference in cost will be more wisely spent in insulating the wall.

FORM COST STUDIES

In connection with the studies of monolithic concrete walls, information was developed regarding form costs and will be briefly discussed.

To anyone familiar with the sliding mold method of constructing hollow walls, it is apparent that the required setting time between courses affects output and costs. The daily output with one set of molds and four-man crew should be at least 400 lineal feet of 9-in. course or about 300 sq. ft. of wall, if the method is to be economical. This output applied to the average six-room house requires about four 9-in. courses per day.

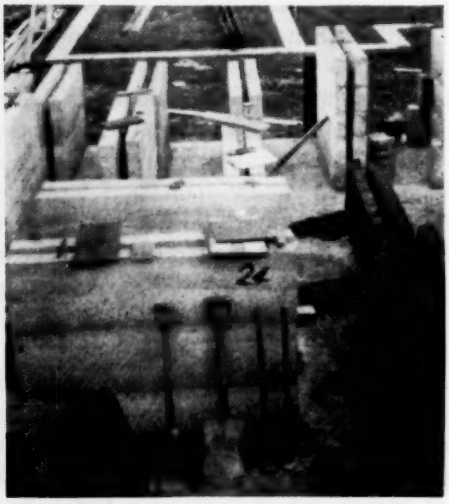


FIG. 7—REQUIRED SETTING TIME BETWEEN COURSES WAS STUDIED ON SMALL WALLS

To investigate setting time a considerable number of experimental walls about 5 feet long by four to seven 9-in. courses high were built. Variables included type and size of aggregate, consistency and proportions of the mix, amount of tamping, air temperature, etc. The

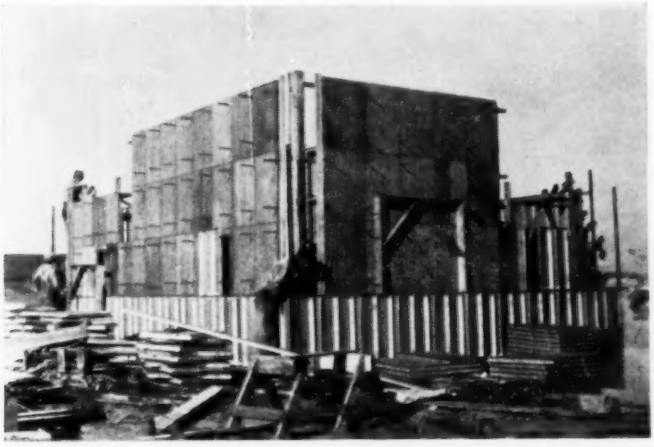


FIG. 8—FORM ERECTION COST STUDY—UNI-FORMS

setting time between the first and second course was made 2 to 2½ hours and was reduced by 15 minutes for each successive course. With optimum combinations of mix, aggregate and temperature the concrete hardened sufficiently in one hour to permit construction of the next course. As many as seven courses were built in one day. Among the important factors affecting setting time were size of aggregate and the air temperature. Required setting time was reduced by increasing the maximum size of aggregate up to 1½ in. Using ½ or ¾-in. maximum size aggregate, the concrete seemed to lack the bridging action apparently obtained with the larger size aggregate and required a longer interval between courses. The effect of temperature may be explained in this way: Whereas one hour setting time was sufficient at temperatures of 90° F., from 2 to 2¾ hours was required at temperatures of 40° F. This study indicated that in so far as required setting time is concerned, at least four courses per day can be built with summer temperatures of 65° F. or above.

Two other types of forms investigated are worthy of mention. These are panel forms, commercially known as Metaforms and Uniforms. While ample cost data regarding their use on large jobs were available, it was desired to obtain actual erection costs based on a specific small house plan.

The forms were erected for 6-in. solid walls with window and door frames installed. Reinforcement and inserts such as required for

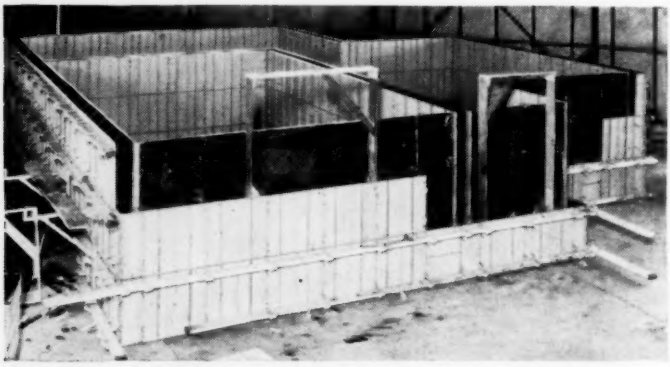


FIG. 9—FORM ERECTION COST STUDY—METAFORMS

joist pockets or floor-bearing, electrical outlet boxes, etc., were omitted. The forms were not filled. Observations were made of their practicability for dwelling construction and the number of man-hours required for erection and stripping. Progress views are shown by Fig. 8 and 9.

The total surface area of exterior and bearing partition walls for which forms were erected was 2,537 sq. ft. Approximately 50 man-hours were required for erection and 16 for stripping. Based on an average wage of 75 cents per hour, the cost of erection and stripping was about $51.00 or 2 cents per sq. ft. of contact area. Total form cost will include in addition, form depreciation and maintenance, spreader ties, trucking charges, miscellaneous items and profit and overhead. These will amount to 6 to 9 cents per sq. ft. of wall. Formwork of this type therefore should range from 10 to 13 cents per sq. ft. of wall, with lower costs being possible with contractors who specialize in this type of work or on large housing projects. With form costs at this figure it is believed that monolithic concrete walls can be erected at a cost comparable with and perhaps even less than for the more common types of wall construction. On the west coast where there is an unusual interest in monolithic concrete houses, good progress is being made in the development of a type of ribbed wall form known as Simplex forms. There are several types of monolithic wall forms which may be readily adapted for use in dwelling construction.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by Aug. 1, 1935.*

Recent Developments in Precast Joist Residence Floor Construction*

BY W. G. KAISER†

MEMBER AMERICAN CONCRETE INSTITUTE

Despite the many admitted advantages of concrete floors for residences, their use up to the last year or two has been limited for the most part to the more expensive homes, doubtless because of their relatively high cost as compared to wood joist floors. The somewhat complicated nature of their construction has also deterred their wider use; at least they presented construction difficulties for the builders who commonly erect small homes.

Obviously if there were to be any considerable increase in the use of concrete floors for residences, there was a need for a type of floor that (1) could be built at a relatively low cost, and (2) which would be easy to build. The concrete joist floor employing either precast concrete slabs or job-placed monolithic slab meets these requirements more fully than any type of concrete floor so far devised. It can be built at a cost only slightly higher than wood joist floors, a price differential so small that the cost usually is not an obstacle. A concrete joist floor is easy to construct. It follows much the same technique of construction as wood joist floors with which every carpenter-builder is familiar.

The popularity of the concrete joist floor is attested by the favorable reception which it is being accorded by architects and builders. They are keenly interested. The home owner, too, seems to want a concrete floor. Much of his supposed prejudice against concrete floors on the grounds that they are cold and hard disappears when he learns that he can now get a concrete floor at a cost only a little higher than for a wood floor. The indications are that the concrete joist type of floor will become increasingly popular for residences of all sizes and in all price ranges.

During 1934, much progress was made in establishing plants and making concrete joists available over a wide area. About 100 plants are now equipped to make joists. This is an increase of approximately

*Presented at the 31st Annual Convention, American Concrete Institute, New York, Feb. 19-21, 1935.

†Manager Cement Products Bureau, Portland Cement Association, Chicago.

500 per cent over the number in business a year ago. The majority of these one hundred plants have made their first installations. Several have built a number of floors, from 10 to 20 each, a remarkable achievement in view of the fact that construction of all kinds was at low ebb. The number of joist plants will probably double in 1935. In addition, there are several hundred prospective manufacturers who are biding the time when there will be a return to normal volume of construction before acquiring equipment.

It is my privilege to present five brief papers relating to recent developments in concrete joist floor construction.

OUR EXPERIENCE WITH TENNESSEE VALLEY AUTHORITY HOUSING PROJECT

BY J. W. WARREN*

It has been our privilege to cooperate with the Architectural and Engineering Departments of the TVA in the development of fireproof, permanent house construction.

While we have furnished cinder block for walls and partitions for more than one hundred houses, which are highly satisfactory, we feel that out greatest achievement has been in precast floor construction. In addition to furnishing floors in all the cinder masonry houses, we also furnished precast floors for several of the houses built of stone, brick veneer, and other construction.

In producing these floors, we used I-shaped joists and precast slabs. A number of the floors were made with cinder slabs, then covered with tarred paper and the hardwood floor nailed direct to the cinder slabs. Most of our precast floors, however, have the surfaces of the slabs exposed. The slabs are approximately 30 in. by 30 in. and 2 in. thick. They are vibrated into molds of our own design and kept in the mold until they have hardened sufficiently to be removed. While most of the slabs were made with red surfaces, some were made with tan, brown, buff and green finishes. The finished floors are smooth and the colors are greatly enhanced by the application of wax in the same way a hardwood or terrazzo surface is treated.

In working with these floors, we developed some novel features of construction which contribute to their practicability. The joist has a mortar groove in the top. The slab has a concave edge, interposed with lugs. The lugs facilitate placing steel in the joints. The effect of grooves, concave edges and lugs is to prevent movement.

*Southern Cast Stone Co., Inc., Knoxville, Tenn.

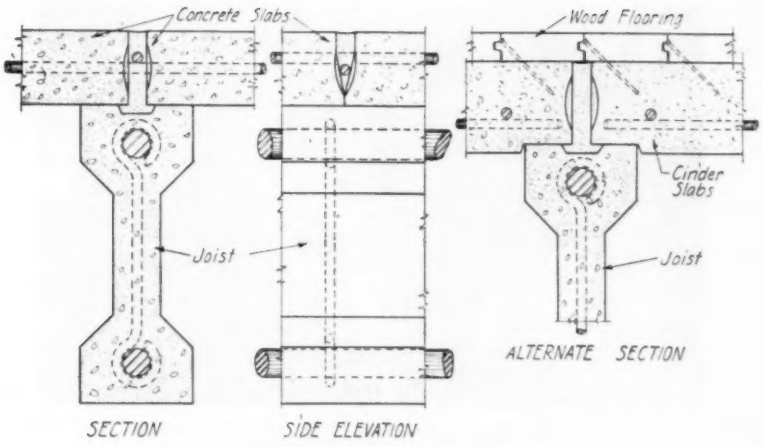


FIG. 1—DETAILS OF JOISTS AND SLABS

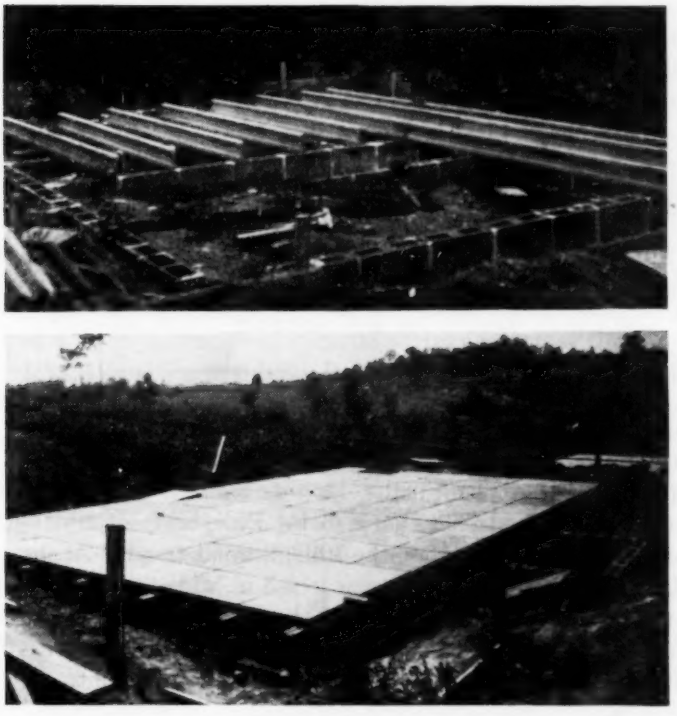


FIG. 2—T. V. A. HOUSES UNDER CONSTRUCTION

FIG. 3—T. V. A. GROUP

Several tests have been made on these floors. The first load test was on a 12-ft. clear span loaded with 80 lb. to the sq. ft. and showed .084 in. deflection. The second test was a 13-ft. span and showed .132 in. deflection. Several tests have also been made by the University of Tennessee, at the request of TVA Engineering Department, and every test has been satisfactory.

As a result of our experience with TVA, we are accumulating a substantial amount of private work in fireproof house construction, and we feel very optimistic of the future in this field.

SUMMARY OF RECENT WORK ON T-BEAMS OF PRECAST JOIST AND FLOOR SLABS

BY F. N. MENEFEE*

Since becoming satisfied that a floor slab can be poured over a 3-in. wide precast joist and that together they will subsequently function as well as a monolithically formed T-beam,[1] the writer became interested to learn what might be expected of 6 in. wide precast beams mortised into floor slabs. It was realized that the intensity of external longitudinal shear on a 6 in. wide beam would be greater, but since it was known that the unrestrained transverse shrinkage of the slab would be twice as great as in a 3 in. wide beam, it was thought that when restrained by the top of the 6 in. beam, the grip might be greater, but if not enough to produce T-beam action, that desirable condition could be brought about by deeper imbedment.

*Professor, Engineering Mechanics, University of Michigan.

[1]JOURNAL AMERICAN CONCRETE INSTITUTE, Sept.-Oct. 1934; *Proceedings* Vol. 31, p. 28, and Mar.-Apr. 1934, *Proceedings* Vol. 30, p. 311.

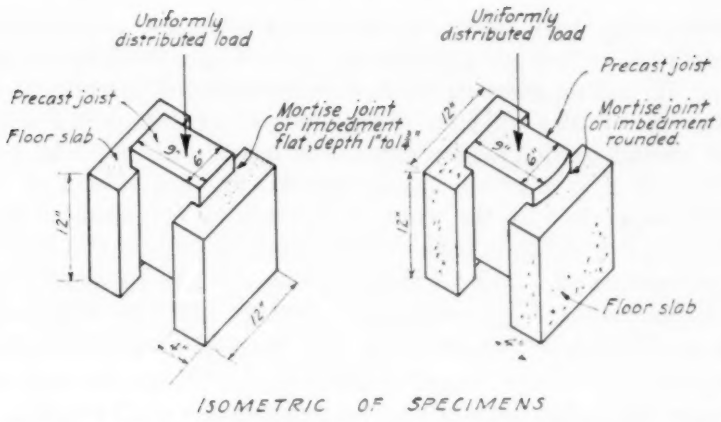


FIG. 4—TESTS SPECIMENS REFERRED TO BY PROFESSOR MENEFEE

For the most part, the writer believes that the bond between the top of the precast beam and the slab cannot be depended upon, although in some joint strength tests with 3 in. joists the concrete of the slab actually bonded to the top of the beam and failure occurred to one side or the other of the plane of the joint. As width increases differential shrinkage between joist (already initially shrunk) and slab would tend to destroy adherence. This leaves the gripping action of the vertical sides of the mortise joint as the medium through which T beam action is brought about unless reinforcement across the joint is provided. The unexpected and rather surprising strength of the imbedded joint arises from the shrinkage of the concrete slab in setting. The precast joist has had its initial shrink and in general will be of a dryer mix so that its total shrinkage across the top will be less than that of the slab immediately above it. To develop sufficient shear resistance from this shrinkage on a 3 in. wide joist, the writer's experience indicates that it is unwise to imbed less than $\frac{3}{4}$ in.

Since shrinkage of the slab goes on for some months after pouring over the joist the resistance to longitudinal slip may be even greater than evidenced by the recorded tests which for the most part are on 28-day specimens.

While as yet not so practicable as the 3-in. wide mortise joint, the tests on 6 in. wide precast joists without reinforcing across the joint may form a groundwork for more extensive experimentation which will prove of greater value.

A floor slab 4 in. thick resting on 6- x 10-in. beams, 16 ft. long, spaced 10 ft. c. to c., and imbedded 1¾ in. with a live load of 100 lb. per sq. ft. and an assumed 36 in. effective width of compressive area in the concrete slab will have about 156 p.s.i. of longitudinal shear to resist around the joint, as compared with 116 p.s.i.[2] for 3 in. beams spaced 2 ft. c. to c. with a 2½ or 3 in. slab and a span of 13 ft. 2 in.

Tests supervised by the writer were made up of specimens similar to those shown in Fig. 2, page 315, March-April 1934, of this JOURNAL, the mixture was 1 : 2.15 : 2.85 by volume, 1 : 2.36 : 2.64 by weight, gravel aggregate. The compressive strength 3200 to 3650 p.s.i. at 28 days. Slump in batches 2 and 3, 4 in., maximum size aggregate ¾ in.

Batch 1. Imbedment intended 1 in., but in specimens 1 and 3 it averaged less due to the dry mix ot the slab and bridging effect of the larger aggregate.

Test Specimen	Age Days	Bond Resistance at Failure p.s.i.	
1	35	95	Slab dry and porous
2	35	114	Slab dry and porous
3	35	88	Slab dry and porous
4	35	131	
5	35	167	
6	35	281	

Batch 2. Beam 3 rounded in joint instead of flat.

Test Specimen	Age Days	Imbedment In.	Bond Resistance at Failure p.s.i.	
1	28	1.25	261	Slab split
2	28	1.44	159	Slab split
3	28	.97	110	Beam imperfect
4	28	1.12	200	
5	28	1.37	220	
6	28	.37	68	

Batch 3. Beam 3 rounded in joint instead of flat.

Test Specimen	Age Days	Imbedment In.	Bond Resistance at Failure p.s.i.
1	28	1.	223
2	28	1.25	362
3	28	1.12	222
4	28	1.37	178
5	28	1.12	339
6	28	1.25	187

The results are promising. In some cases (those running over 300 p.s.i.) the resistance is high enough to justify more experimentation. With mixtures that will flow readily down the sides of the beam to the full and intended depth of 1½ to 1¾ in. mortise, it is believed

[2]JOURNAL AMERICAN CONCRETE INSTITUTE Mar.-Apr. 1934; *Proceedings* Vol. 30, p. 311.

tests will show results justifying the use of the precast 6 in. beam if other factors encourage its use. Reinforcing will in this case unquestionably place it within the bounds of safe construction.

Some of Our Experiences in Concrete Joist Floor Construction

BY G. C. TURNER AND HERMAN FRAUENFELDER*

In introducing the precast concrete joist floor in the St. Louis market, we found from the very outset, architects, contractors and builders sincerely interested. We had difficulty, however, in getting a contract for our first installation because contractors, unfamiliar with this new type of construction, were making their bids too high, for which they could not be seriously criticized.

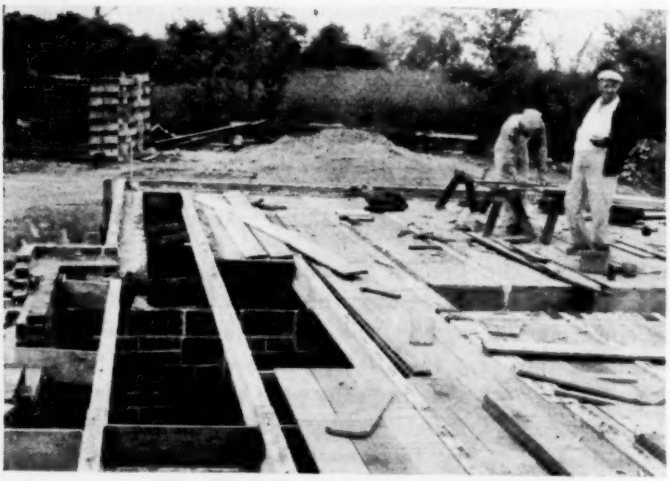
We met this difficulty by ourselves taking the contract for the concrete first floor of a "display home". We manufactured and furnished the concrete joists and sub-let the concrete work to a concrete contractor. The job proved to be an effective means of bringing this type of construction to the attention of the public. More than 50,000 people visited this demonstration home and considerable interest was centered on the precast concrete joists and concrete floor. Since the opening of this house in October, we have sold 15 additional jobs, many of them, we believe, a direct result of this first installation.

We find that service must be given to the architect or builder. The architect specifies concrete joists with thickness of floor slab and also shows the sizes of steel or concrete supporting beams on his plans. He does not furnish a joist framing or setting plan. In submitting prices for the required concrete joists together with the necessary reinforcement for the slab, the asphalt paper for lining the form boards, we submit a complete framing plan. Figuring a job in this way serves a two-fold purpose: (1) we determine the required number of joists and amount of slab reinforcement to insure good construction; (2) the contractor receives from one source the three specialties used, i. e., precast concrete joists, slab reinforcement and asphalt paper lining.

With a setting plan and information as to joist and reinforcement requirements at hand, the contractor can readily make up his bid for the floor.

Before preparing the framing plan, we confer with the heating, plumbing and electrical contractors to determine where openings in

*Marcrome Art Marble Co., St. Louis, Mo.



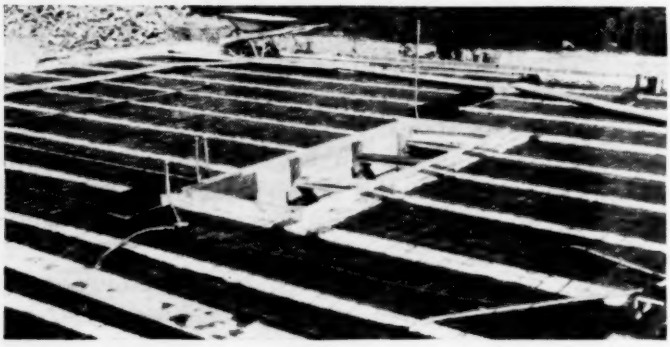


FIG. 5—BUILDING FORM BETWEEN JOISTS FOR SLAB

FIG. 6—JOISTS PLACED, FORM BUILT, STEEL IN POSITION, CONDUITS AND FURNACE DUCTS IN POSITION—READY FOR CONCRETE SLAB

the slab to accommodate their work are to occur. These are indicated on the framing plan and submitted to the architect for his approval.

The joists are manufactured and marked as shown on the framing plan so that the contractor knows exactly where each is to be placed. Jobs handled in this manner have worked out without difficulty.

Special Construction Details

We have found that it is advantageous to use joists at the edges of parallel walls even though it increases the number of joists in the job.

On one job where a room was built over a garage, it was necessary to provide economical insulation. This was done by attaching wooden bases to the joists to which the insulation could be nailed. Boards were cut to the same width as the steel pallet used in the joist mold in place of the pallets. The boards were fastened to the tension rod in the joist.

By working with the heating contractor, we showed him he could save on the return cold air duct by utilizing the space between joists. In such cases, we attached wood bottoms on the joists to which sheet metal could be nailed, thus forming a duct at less cost than by building it entirely of metal.

Construction Advantages

In practice, we have eliminated shoring except on long spans. This is a distinct advantage over other types of concrete floor construction from the standpoint of economy, and the contractor is always sure of level forms regardless of the condition of the soil in the basement. He does not have to worry whether his shoring is sinking during construction. This point was a deciding factor in giving us one of our largest jobs.

One builder tells us he likes this type of floor construction because shrinkage is eliminated and it isn't necessary for him to return within a year to renail or reset base and toe boards and there are fewer cracks in plaster in partition walls caused by shrinkage of wood joists.

Another builder points out that the publicity and reputation gained by constructing concrete floors with precast concrete joists are of greater value to him than the small difference in the cost of such floors over wood construction.

Tests of Precast Joist-Precast Slab Floors

BY R. E. COPELAND*

The structural and economic qualities of floors built of precast joists and precast slab members depend to a considerable degree on the strength of the joint between the joist and slab. Adequate interaction between the two members so that the slab may be safely counted on to take the compressive stresses is essential to an efficient design and maximum strength and rigidity.

The effect of different types of joints on the structural performance of floors of this type is being studied by the Development Department, Portland Cement Association, in a series of tests involving test panels 14 ft. and 20 ft. long by 4 ft. and 8 ft. wide. The panels are uniformly

*Development Department, Portland Cement Association, Chicago.

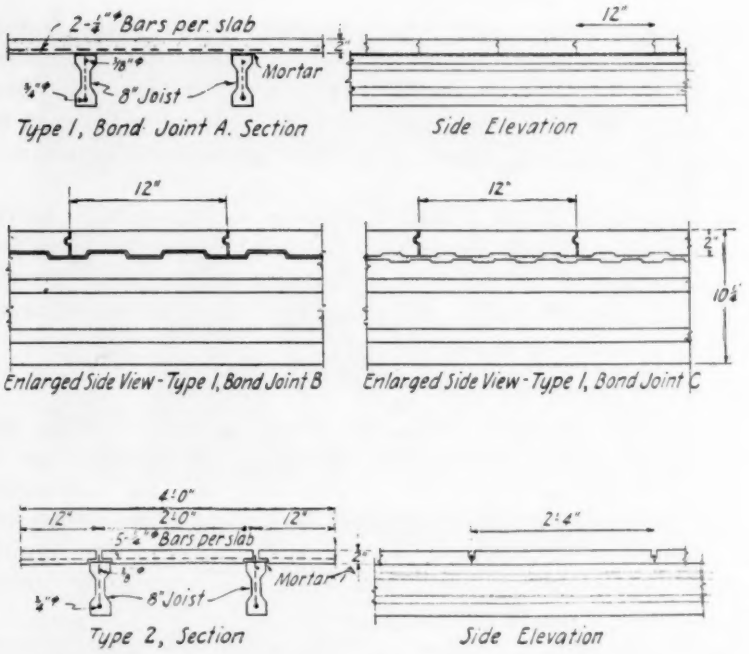


FIG. 7—PRINCIPAL DETAILS OF SLAB TYPES 1 AND 2 AND ALTERNATE BOND JOINTS USED WITH TYPE 1

loaded until failure occurs at the bond joint or other part of the construction. Test methods closely follow those used by P. M. Woodworth and the author in some tests described in the A. C. I. JOURNAL.[1] Some of the significant results of the 16 tests of this series completed thus far may be briefly discussed at this time as they apply to panels 14 ft. long by 4 ft. wide. At a later date, a more comprehensive record of the study will be offered for publication in the JOURNAL.

Fig. 7 shows the principal details of slab types 1 and 2 and the alternate types of bond joints used with slab type 1. Other variables studied include composition and strength of the mortar used for the bond joint, type of aggregate and roughness of the bonding faces.

To afford direct comparisons, the joists were reinforced with the same bar sizes as used in the previously reported tests of precast joist and cast-in-place slab construction. The depth from the top of the slab to the tension steel varied slightly with different panels but averaged around 8.92 in. Assuming fs = 20,000 p.s.i. the resisting moment of the steel translates to a total design load of 122

[1] "Some Tests of Load Capacity of Floors Made with Precast Concrete Joists" by R. E. Copeland and P. M. Woodworth, JOURNAL AM. CONCRETE INST., Mar.-Apr., 1934; *Proceedings* Vol. 30, p. 311.

lb. per sq. ft. Deducting the weight of the test panel the superimposed load producing a unit tensile stress of 20,000 p.s.i. is 90 lb. per sq. ft. As will be indicated later, the ultimate loads sustained by several panels developed the tension steel well beyond its yield point resulting in a tension failure.

The panels built with slab type 1 and bond joint *A* which had plain bonding faces gave the poorest results; the joint rupturing with superimposed loads of 40 to 160 lb. per sq. ft. Results in this group were greatly affected by the texture of the bonding faces. The rough finished surfaces produced strengths up to four times that obtained with the smooth surfaces.

In bond joints *B* and *C*, slab type 1, a mechanical bond is obtained and load capacities were greatly increased. Panels with bond joint *B* failed with superimposed loads ranging from 278 to 288 lb. per sq. ft.; with bond joint *C* 218 to 285 lb. per sq. ft. Where the ultimate load exceeded 240 lb. per sq. ft., failure generally was due to a tension or bond failure of the joint reinforcement. Bond joint *B* performed particularly well, there being no indications of joint failure at ultimate loads with panels having this type of joint.

Very satisfactory results also were obtained with the two panels embodying type 2 slab. Failure in both tests occurred with a superimposed load of 280 lb. per sq. ft. and was ascribed to tension failure of the joist steel. In these panels the shear reinforcement in the joists consisted of looped stirrups, the loop projecting about 1 in. above the joist. The embedment of this loop in the mortar fill between the slabs apparently added substantially to the joint strength.

Deflections at mid-span of the floor panels ranged from 0.05 in. to 0.43 in. with the 80 lb. per sq. ft. loading. Panels with joint types *B* and *C* slab type 1, and panels with slab type 2 deflected between about 0.10 and 0.20 in. with this loading.

Tests remaining to be made include one panel of 14 ft. span with bond joint *C* and concentrated loading and one panel of similar span but 8 ft. in width, in which we will study the effect of concentrated loads on the distribution of stresses in the slab in an effort to obtain a basis for the more efficient design and reinforcement of the slab members.

The test results obtained to date indicate that slab type 1 with bond joints *B* and *C* and slab type 2 with bond joint described produce a substantial degree of continuity between the slab and joist with the result that floors thus built develop a rigidity and strength which is considered adequate in residence and similar light load

structures. There may be, of course, other types of joints which will produce equally good or even better results. The mechanical bond or tie appears to be an important feature of such joints.

THE LITH-I-BAR PRECAST JOIST AND SLAB

BY C. V. BERRY*

It is generally recognized that the building industry has been slow in the development of cheaper and better methods of construction.

We feel that much progress in this direction has been accomplished by the advent of precast structural floor and roof units. Lith-I-Bar construction embraces precast concrete joists and slabs, as well as accessories which go with them to make a complete precast floor or roof. A special feature is the bonding of the slab to the joist. From an engineering standpoint, this bonding of slab and joist can best be explained by the fact that we use factory-made concrete with a carefully controlled water-cement ratio, which is rolled and kneaded in the molds under a pressure of approximately 2000 p.s.i., preventing accumulation of laitance on the bonding surfaces. As a result, bonding areas present positive gripping surfaces and are not mere skin coats of accumulated cement and fine particles. It is interesting to note in this connection that tests show that our bond strengths run as high as 600 lb. per sq. in., in shear.

Granting that a reputable concern is manufacturing the precast joist and slabs, the engineer on a project can safely assume that he is

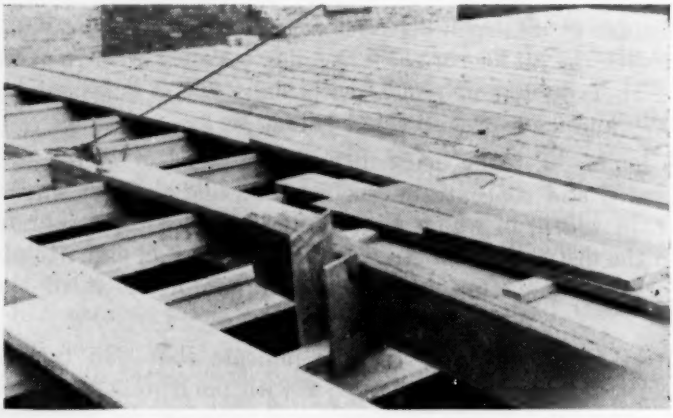


FIG. 8—PRECAST JOISTS AND SLABS—LITH-I-BAR

*Kalamazoo Haydite Tile Co., Kalamazoo, Mich.

FIG. 9—UNDERSIDE PRECAST JOIST AND SLAB FLOOR

working with a product of known and uniform quality, because ingredients and manufacturing operations are all carefully controlled at the plant.

Hundreds of installations have been made in many types of buildings and conditions during the last five years. Floor loadings have varied from residence loads of 40 lb. to warehouse loads of 600 lb. per sq. ft.; the spans from 8 ft. to 27 ft.

Precast joists are used either with job-placed or precast slabs. Insofar as the job-placed slab is concerned, its chief economy lies in the simplicity of the form system required, along with the saving in reinforcing steel due to relatively light dead weight load of this construction. Briefly, the form system consists of wood panels supported on cleats which in turn rest on the bottom flanges of the joists. No shoring is required except in the center of long spans.

On a 20-ft. span, and a live load of 100 lb. per sq. ft., the combined weight of joist and slab is only 22 lb. per sq. ft. These units can be entirely erected with non-skilled labor and the cost of labor does not exceed 3 cents per sq. ft. for placing both units.

The mechanical trades and partition work can start as soon as the floor above is covered, as there is no cumbersome shoring in the way.

Pitched roofs are seldom built of poured concrete because of cost and weight, so that many buildings are fireproof to the roof line,

FIG. 10—LITH-I-BAR ROOF CONSTRUCTION

have wood or combination wood and steel roof — a compromise necessitated by old concrete methods and high costs.

The joists are laid up on a pitch as are wood joists or rafters. The ends are butted to a shoulder on top the outside wall with tie rods running from one wall to the other. The ridge is blocked in with concrete, the slabs are laid over the joists and joined to same with grout and the job is done. Incidentally, nailing concrete can be used in the slab, and slate, copper or asbestos shingles applied for the surfacing. Roofs with spans up to fifty feet can be covered in this way.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by Aug. 1, 1935.*

Architectural Concrete Makes Prefabricated Houses Possible*

BY JOHN J. EARLEY†
MEMBER AMERICAN CONCRETE INSTITUTE

The concrete industry is turning its attention from major construction to small houses. Our industry likes to think that its interest in small houses has roots in the past, and that it might have contributed much to progress in this field had it not been so completely occupied in another. Let that be as it may. The fact remains that the concrete industry has contributed almost nothing to the construction of small houses.

How small the contribution has been may be illustrated by a survey made in 63 cities by the Real Property Inventory Unit of the United States Bureau of Foreign and Domestic Commerce. It shows that of 1,728,521 dwellings, 1,404,456 were built with wood, 186,167 with brick, 114,522 with stucco, 7,229 with concrete and 4,277 with stone. The remainder were built with adobe sun-dried brick and other materials. Concrete stood in fourth place, but it represented only four tenths of one per cent of the total number.

The quality of the concrete used in these houses is not representative of the real possibilities of the material. The work was done almost entirely by unskilled workmen. The technicians of the industry, who have made the design of concrete a science and its control an art, had not yet applied their talents to small houses.

Great social changes are taking place in the United States. The people demand that further steps be taken towards a social justice that will effect a more general and equitable distribution of the comforts of living.

INDIVIDUAL HOMES ASSURE SECURITY

Everyone is seeking security. Labor has always sought security but has not achieved it. Capital has assumed security and is now in danger of losing it. The present social movement is a levelling one and it is entirely possible that we all will come to understand that the security

*Presented at the 31st Annual Convention, American Concrete Institute, New York, Feb. 19-21, 1935.
†Architectural Sculptor, Washington, D. C.

which we desire for ourselves and our dependents, lies in the nation's ability to provide food and shelter for everyone. It seems to me that the simplest way in which such security can be achieved is to enable everyone to procure a small house and a plot of ground, which can be cultivated and which will produce sustenance.

The development of the mechanical sciences has been the glory of the past one hundred years. Much of this progress has contributed to the comforts of living. Improvements have been made in lighting, heating, sanitation and in a multitude of labor saving devices, which can best be utilized only behind the protection of genuinely modern walls.

But the benefits of that development have been restricted to a limited few. The present movement for social justice is a protest against such restriction and an effort to distribute these benefits to all. The success of that effort, the practical application of the social sciences, promises to be the glory of the coming years. Out of it will come a new American home, the finest house of any age, and it will be within the reach of every family.

THREE DIVISIONS OF SMALL HOMES

Recent investigations of economic conditions in the United States classify small houses, that is one family houses, in three broad divisions.

In the first are houses costing less than $3,000 and used by families who need homes that can be purchased in small installments with or without the assistance of the government. It might even be part of the solution of the social problem that the government should pay the entire price and give houses to these families. The houses of this class provide simply shelter. Much that is desirable of beauty and comfort must, for the present, be sacrificed to low cost. These houses can be built by mass production either by private enterprise or by the government.

The second division contains houses costing from $3,000 to $7,000. They add to shelter the essentials of comfort. They would be occupied by active workers who constitute the backbone of national industry. From the point of view of the concrete industry, this division would probably be the most fertile, as well as the most interesting field, because it affords greater opportunity for the expression of individual requirements than do government specifications.

In the third classification there is added to shelter and essential comforts something which approaches luxury. The cost would be from $7,000 to $12,000. In a few instances, it might be higher. Operations in this field would be fewer in number but would afford greater

opportunity for the display of design and craftsmanship. These houses would be occupied by those who hold preferred positions in industry, commerce and government, who appreciate the finer things of life, and who can afford the comforts of fine living.

NEW MATERIALS MUST PROVE WORTH

Let not those who plan to go from the field of major construction into that of minor building think that the latter field is unoccupied, or that the public is waiting to welcome them with open arms. On the contrary, the field is well filled and those who enter from major construction will be regarded as innovators with new materials, who must win their way and prove the worth of these materials.

In all parts of the United States, in cities, towns and countryside, are the builders of small houses. They have built the houses in which our people live. They are able and prepared to build more houses. Their methods and materials are highly conventional. For many years nothing new has been done. Indeed it would seem that the men who build small houses have almost completely escaped the influence of the new materials and methods, which in recent years have revolutionized major building. The barrier of cost has been thoroughly effective.

Reinforced concrete and steel are to them new materials. Their use would be an innovation and upon the innovator rests the obligation of either reorganizing the whole small house industry in the face of bitter, desperate opposition by the present small house builders or of grafting these new materials and methods onto the industry without disturbing its normal processes. Having considered this problem from its own particular point of view, our studio has decided to graft its material and processes onto the small house industry without disturbing it more than is necessary.

NEW FIELD FOR CONCRETE INDUSTRY

For the next few years the concrete industry and the architectural profession will find occupation in the building of small houses. The problem affords full scope for the ingenuity of designer and craftsman. No material is excluded. There is no limitation of form or size. Cost is the only limiting factor. A small house must not cost more than a family can pay for it. If it is to be sold on the installment plan, if its cost is to be amortized in 12, 15 or 20 years, these installments must not exceed the rent which the families who are to buy these houses are now paying.

By a policy of Government-insured mortgages, the Administration at Washington is talking about promoting the building of from half a million to a million new homes a year. Competent observers, however,

say this will be possible only if there is a radical decrease in the cost of building operations. So far, there has been little decrease in the cost of building materials or building wages.

There is a field of building construction, however, in which major economies are being achieved. That is the field of prefabricated houses. They might better be described as houses assembled out of prefabricated units. By adding the savings of mass production to the achievements of science, industry is producing prefabricated kitchen units, heater units, bathroom units, plumbing units, partition units, and prefabricated exterior walls, that promise to give to the home owner of tomorrow an assembly of comforts and luxuries which should give him for $5,000 what might have cost $10,000 a few years ago.

This new art of prefabricated houses calls for a new school of design, a school of design in which the advantages and the limitations of the assembly of such prefabricated units will be handled by men who know how to make the most of the new materials at their command.

WALLS DEMAND PERMANENCE AND BEAUTY

In such designing, the use of prefabricated exterior walls becomes a vital problem. And if prefabricated interior units, with more or less fixed design, are to be assembled, into a usable home, it is doubly important that the exterior walls give to the assembled product a permanence and a beauty which will attract the home owner who is to live in it.

Design is properly the contribution of the architect. It is his function to plan and compose in the spirit of the time, with all the materials available. In the case of the new American home of tomorrow, these materials will include all the various prefabricated units which Science and Industry are placing at his command.

An architect who knows these materials—both new and old—and who can plan and compose such a home in the spirit of the time in which we are living, will contribute much to the construction of small houses. In fact, their acceptance by the general public will largely depend upon him. Public interest establishes the place of the architect in this field, because good appearance and timely styling are real values.

Such a man must not only be a creative artist of the first order but he must live the life of the people. He must belong to his generation. He must know its thoughts and desires, and sense the social changes that are taking place. He, of himself, without undue labor or affectation of any kind, will intuitively do the things which the people desire.

Execution of design with an open mind and a skilled hand is the contribution of the craftsman. Work in an architectural studio for

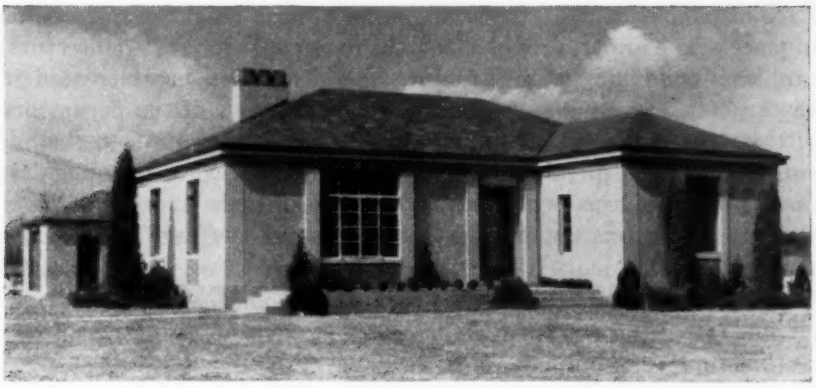


FIG. 1—FIRST OF THE EARLEY CONCRETE HOUSES

many architects on many projects is conducive to an open mind. One soon learns that a church need not be Gothic: It can be in some other style just as well. It need not be stone: there are many other suitable materials. But it is exceedingly important that the craftsman be prepared to execute any design in a proper material with understanding and sympathetic skill.

MOSAIC CONCRETE AS ARCHITECTURAL MEDIUM

In the most interesting period of the life of a great people when the old conventions are being broken and new forms are sought, our studio has been favored by the opportunity to execute the ornamentation of their buildings in and out of conventions, with old and new materials, but always with fine workmanship and close attention to economy. In this we have developed a new architectural medium, which we call mosaic concrete. It is our contribution to the construction of small houses. We know it very well. We are not deceived about its value relative to other materials, nor about the probable extension of its use. It will make strong and beautiful walls, which we are sure will receive popular approval. They can be prefabricated with high perfection and great economy and they can be assembled in place by any practical builder.

Among the members of the American Concrete Institute, many have something to offer to the construction of small houses. For the present all should approach the problem independently and suggest such solutions as will be prompted by their own technical ability or the characteristics of their material. Our experience in the execution of many interesting problems indicates that our type of architectural concrete may best be applied to small houses in the form of thin precast

slabs or panels, made in the studio, taken to the building, and erected in place. The importance of thin slabs to our solutions of architectural problems combining both structure and decoration has increased in the last ten years. We were not entirely conscious of the persistence of this tendency until current work directed our attention to the advantages of such prefabricated panels. We now realize that our first mosaic concrete ornaments were precast in thin slabs and that most of the important ornamentation on all our works has been done in the same manner.

The thin precast slab is, therefore, a logical and necessary development. It is the best application yet made of concrete to decorative architecture. This is apparent on the dome of the Baha'i Temple, which has been presented to the Institute, and on the mosaic concrete ceilings of the Department of Justice, which have not been presented.

In order to demonstrate the use of precast panels in the walls of small houses and to gain experience for ourselves, we have built in the suburbs of Washington on the Colesville Pike, a small house. It is a one story house of the apartment bungalow type with five rooms and bath. It contains 15,000 cubic feet and belongs to the third classification mentioned above, that of houses costing between $7,000 and $12,000. It is thoroughly insulated and contains all modern equipment.

THE POLYCHROME HOUSE AND THE PUBLIC

Many members of the industry have asked us why only the walls and the foundations of this house are made with concrete. The answer is simple. We did not deem it wise to go too far, by building our first house entirely of concrete. That may come later. But we must first ask the public whether it likes our little polychrome house with its beautiful mosaic concrete walls, which require no maintenance and which are permanently beautiful, before we ask whether it would like a roof of the same material.

We also hesitated to do more because we have found that many people who have had an ingenious idea for the building of a small house with some special material have permitted their idea to obsess them. They have attempted to build the whole house with one material, and have fallen into the pit of misapplication which lies across such a course. We think that the new American home will be a composite of materials, each used to the best advantage for the good of the house. We think that it will be made from prefabricated parts and assembled in the field by the men who are now the builders of small houses.

The advantage of prefabricated panels lies in the almost perfect technical control which can be exercised over their fabrication in a shop

or studio, in the ease with which they can be shipped and assembled and in the economies which they afford.

HOW PRECAST SLABS ARE MADE

In making the concrete for our thin slabs, materials for aggregate are selected for color and strength. They are crushed and screened to predetermined sizes, mixed with cement and water, and cast in a mold. When the cast has hardened, but before it has thoroughly set, the mold is removed and the surface of the cast is brushed with steel wire brushes until the aggregate has been evenly exposed over the whole surface. Subsequently the surface is washed with diluted muriatic acid to bring out the full brilliance of the aggregate. All of which has been thoroughly discussed before the Institute and needs no repetition.

The slabs for our small concrete house are cast face downward in flat shallow molds, which are open at the back and which afford unimpeded access to the materials in the mold. These molds are about 9 x 10 ft. and but 2 in. deep. It is therefore easy to extract from the concrete, by absorption, the excess water used in the mix, as a vehicle to carry the inert particles into place. This method was discussed at the 1934 convention of the Institute in a paper describing the precast slabs for the dome of the Bahai Temple.[1]

Additional water may be extracted from apparently dry concrete, by a second application of absorptive material accompanied by vibration. It is interesting to note that the application of vibration to concrete, from which most of the original water has been extracted, will withdraw still more water and will not disturb the granular arrangement of the aggregate. Segregation often occurs when concrete, containing aggregate of more than one size and much free water, is vibrated, but it does not occur when enough free water has been previously extracted from the concrete to destroy its mobility.

We consider this variation of technique another step towards the production of concrete with only capillary water in the mix at the time of set. Such a condition has been produced heretofore only by mechanically applied pressures so high that their application to large thin slabs seems both impractical and uneconomic. Further I am convinced that sufficient water can be removed from cast concrete slabs by absorption and vibration and without mechanical pressure to produce concrete which will meet all the requirements of architecture.

NEED FOR MORE PERFECT CEMENT

In the studio our control of concrete, by controlling the water and the aggregate, reveals the characteristics of different cements. And

[1] "Architectural Concrete of the Exposed Aggregate Type," by John J. Earley. JOURNAL Amer. Concrete Inst., March-April 1934, *Proceedings*, Vol. 30, p. 251.

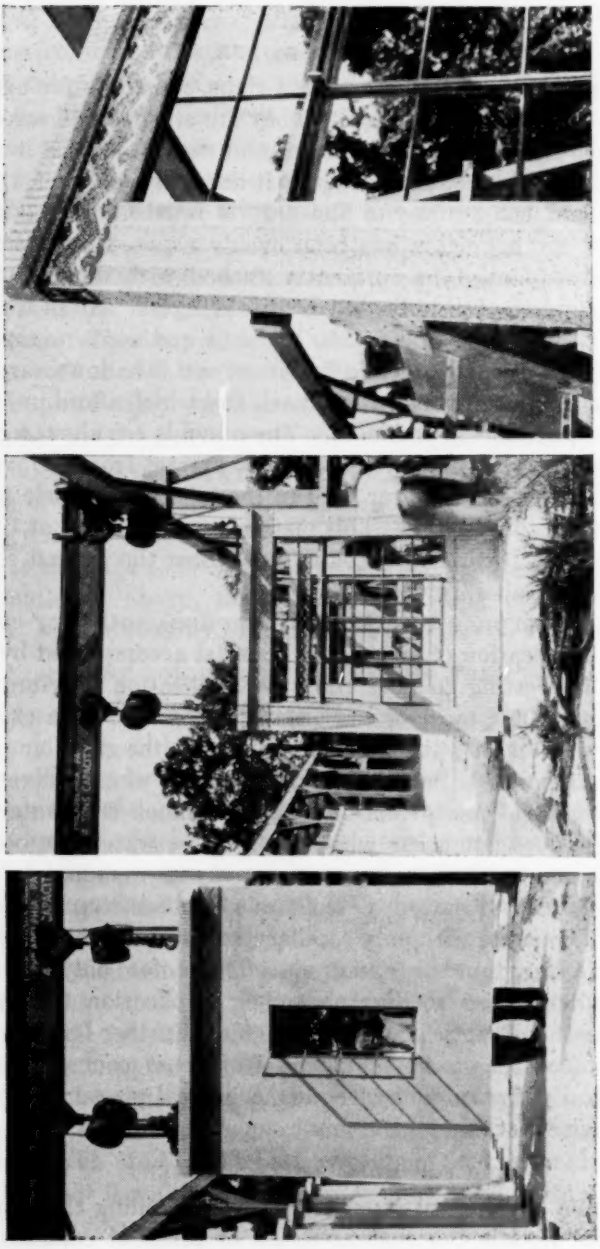


FIG. 2, 3, 4—HANDLING PRECAST CONCRETE MOSAIC HOUSE SECTIONS AT EARLEY PLANT, ROSSLYN, VA. METAL CASEMENT WINDOWS ARE CAST IN THE SLABS

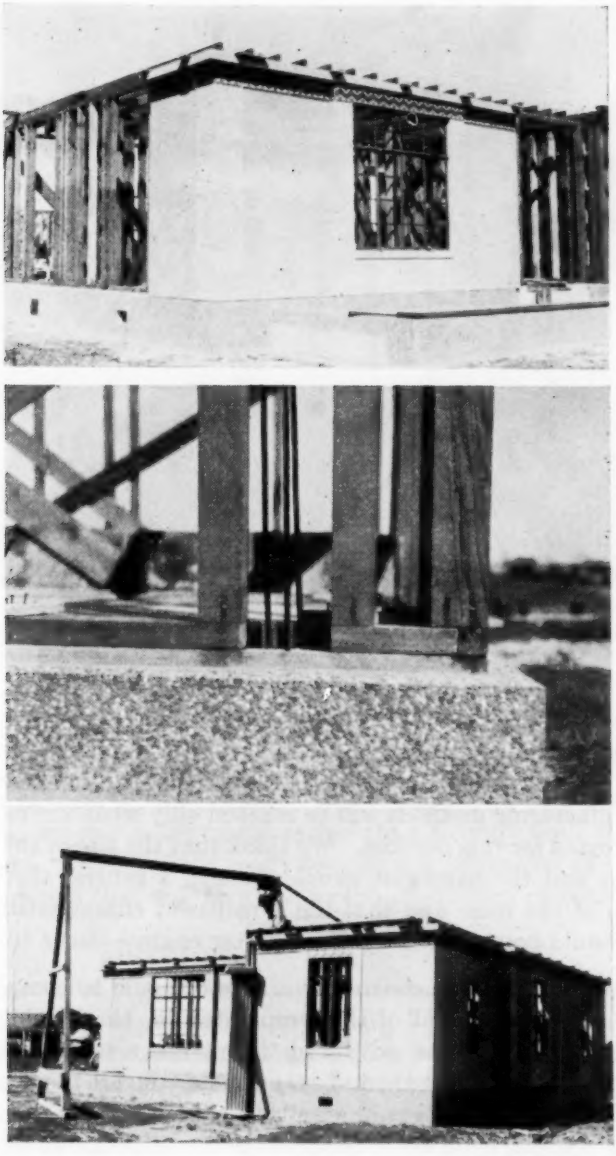


FIG. 5, 6, 7—EARLEY HOUSE UNDER CONSTRUCTION

In center picture note weather-stop ledge of cast-in-place foundation. Also the verticle rods between two studs. In this space a cast-in-place stud ties the wall slabs in a joint that is not pointed. Backed up with foil on the inside the slabs at this stud are tied by steel loops (cast in the slabs) and made flexible by a short piece of rubber tubing at the base of the loop where it is embedded in the stud.

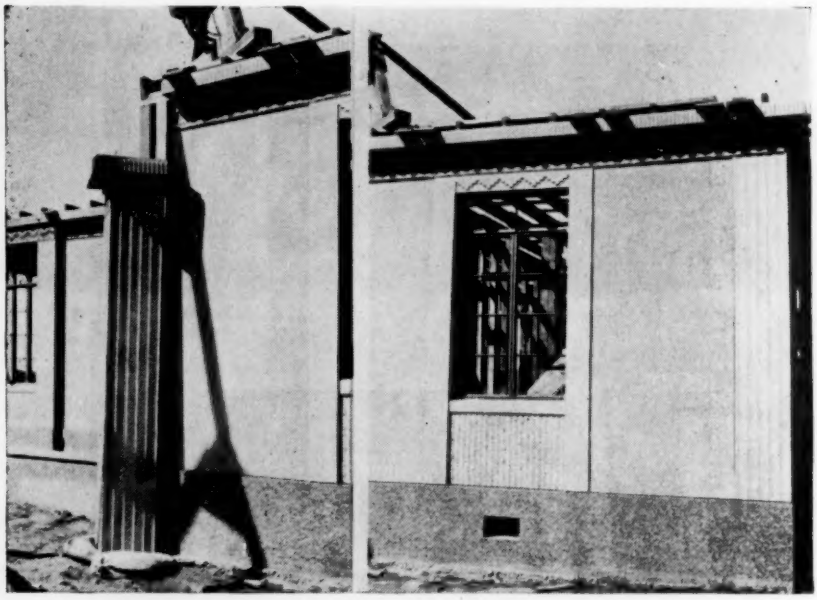


FIG. 8, 9—CONSTRUCTION VIEWS—CORNERS ARE SEPARATE PIECES

our experience in the fabrication of thin slabs for small houses indicates that the highest perfection of the material and the fullest economy of new manufacturing methods will be reached only when the cement has been perfected for this purpose. We think that the size of the particles of cement and the aggregate should fit into a general theme for the gradation of the mass and that the constituent characteristics of the cement should be modified to afford better control of the time of set.

Any article made with concrete in a studio should be completed in a working day, independent of the temperature or the humidity of the air. Recent research and advanced theory make such modifications possible, and I think that the industry should convert them into facts. It is true that such changes in standard cement are not generally regarded as necessary. This is because the manufacture of concrete in the field is not yet under sufficiently perfect control to make the need of better cement generally apparent.

But if we had available such cement we could devise processes of fabrication which would produce, with concrete, the most beautiful masonry walls in the world at a lower cost than with any other material.

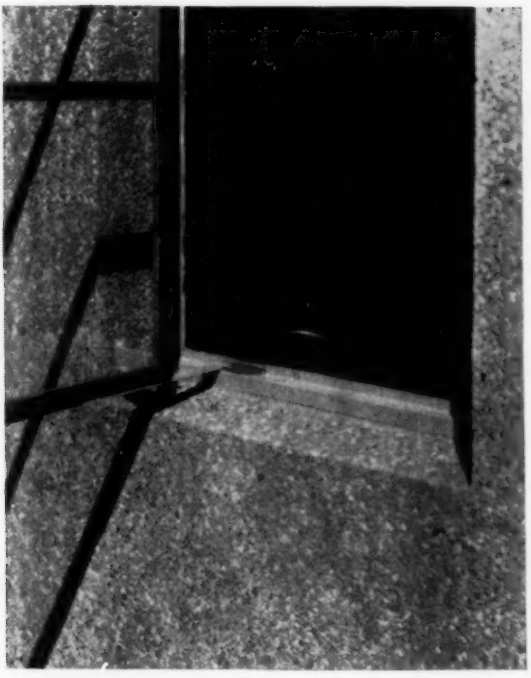


FIG. 10—CLOSE-UP AT A WINDOW SHOWING SURFACE TEXTURE

It might be wise for the manufacturer of cement to face this problem of making special cement now rather than to wait until the need is paramount. Our studio is ready to take full advantage of every step in the progress that will be made in the production of better cement.

HOW SLABS WERE SHIPPED AND MOUNTED

Another value of prefabricating thin slabs is found in the ease with which they can be shipped and erected. Thirty-two slabs constitute the walls of our first concrete house. They were hauled ten miles from our plant at Rosslyn, Va., by a 1½-ton truck and were assembled in the walls of the house directly from the truck. A 1-ton chain hoist with traveler and a short piece of six-inch steel beam, set on "A" frames, were all the equipment needed. Such equipment can easily be obtained in any town or city and can be operated by any building mechanic.

The "A" frames were made with 4 x 4-in. sticks. The shorter frame rested on the rafters, the other and longer one rested on the ground.

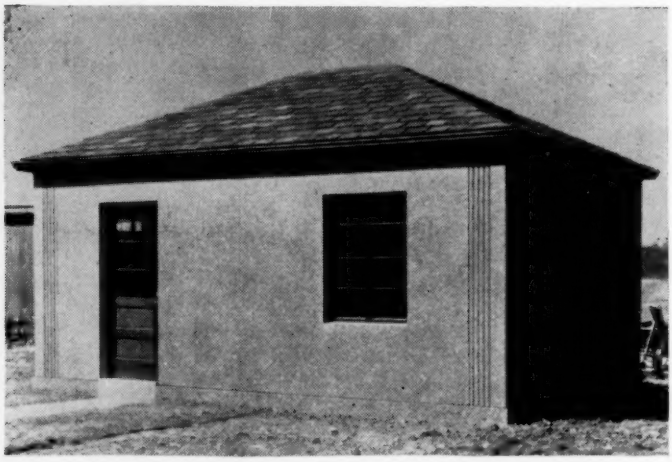


FIG. 11—THE GARAGE IS ALL IN MOSAIC CONCRETE OF BUFF QUARTZ

The truck was driven under the beam and the slabs were unloaded and set by two men, who also moved the equipment when necessary.

After setting the slabs, they were fastened together at the joints, supported and tied to the foundation by small reinforced concrete columns. A special device, which attached them to the columns, allowed them to move in any direction independently of one another. The device consisted principally of dowels protruding from the back of each slab along the vertical edge where the columns closed the joint. Around each dowel was a piece of thin rubber hose about 1½ in. long. The concrete columns held the dowel firmly excepting the portion which was in the rubber hose. That portion remained free to form an elastic support. Each slab, therefore, was supported by a number of flexible steel rods and was free to move with the expansion and contraction of the concrete.

This principle of construction—the free attachment of prefabricated thin slabs to a reinforced concrete skeleton—is another step forward in the development of concrete as an architectural medium. It was suggested by the small house problem, but the principle is larger than the problem itself and we may expect to see it applied to major construction when major construction will again become important.

Prefabricated thin slabs used for forms for structural reinforced concrete, as was done for the ceilings of the United States Department of Justice building at Washington, or flexibly attached to reinforced concrete skeletons as in our small house, offer an architectural medium,

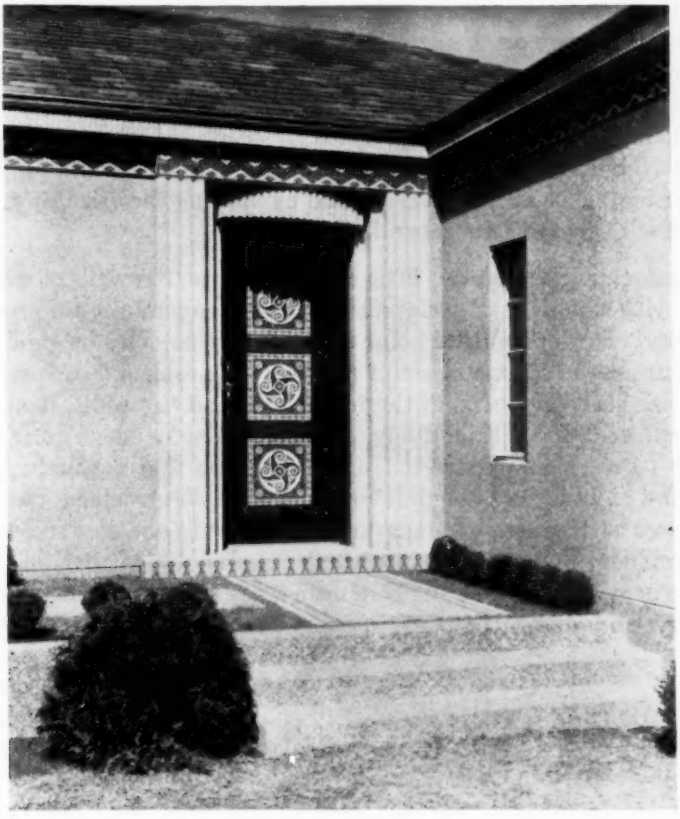


FIG. 12—ENTRANCE DETAIL—MOSAIC PANELS IN THE DOOR

the like of which has never been known, and the limits of which are the limits of human ingenuity.

WIDENS OPPORTUNITY FOR ARCHITECT

There is a prevailing opinion that prefabrication and the mass production of small houses is hostile to the individualist and the architect. This is not true, and it is particularly not true of houses made with mosaic concrete panels. This material is so facile, because of its plastic nature, that a great degree of individuality can be maintained without sacrificing the economy of manufacturing processes.

All this has been accomplished by precasting the panels in molds, into which are built all the elements of excellent craftsmanship neces-

sary to the proper execution of architectural design, and which are so flexible that they are susceptible to wide variations. It must not be understood that because mosaic concrete houses are cast in molds, they are necessarily identical. That is not so. Great individuality may be obtained from a single set of molds by using different aggregates to change the color of the panel, by changing the ornamentation, by recomposing the parts of the house, and by varying the orientation of the houses with regard to a general scheme.

In presenting to you this little concrete house, the walls of which are made with thin slabs of prefabricated mosaic concrete, I am presenting a reality. It is a beautiful home in which anyone may be glad to live. It is our answer to one phase of the housing problem. Although concrete has been used only in the foundation and the walls, it may well be used much more extensively. Other parts of a house offer opportunity to reinforced concrete. It is our hope that the industry will continue to work with the problem until it has been solved, until there is a house with concrete foundations, frame, walls and roof so beautiful, so permanent and so economical that when equipped with all modern aids to comfort it will be the New American Home, the finest house of any age, and within the reach of every family.

For such discussion of this paper as may develop, readers are referred to the JOURNAL, *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by Aug. 1, 1935.*

Placing Concrete By Means Of Vibration

Introducing the work of A. C. I. Committee 609, Vibration of Concrete

Editor's Note—*In the following pages, papers by* M. O. Withey, Thomas E. Stanton, Jr., Lewis H. Tuthill *and* F. H. Jackson *continue the series presented at the 31st Annual Convention, New York, February 19-21, 1935, by Committee 609, Alfred E. Lindau, Chairman. The Chairman's introduction to the series and three of the papers—by* C. M. Hathaway, F. V. Reagel *and* W. R. Johnson, *were published in the March-April* Journal. *The series will be concluded in the next* Journal *(Sept.-Oct.) with brief papers by* Ben Moreell, Sam Comess *and* T. C. Powers.

Freezing and Thawing, Permeability and Strength Tests on Vibrated Concrete Cylinders of Low Cement Content*

BY M. O. WITHEY†

MEMBER AMERICAN CONCRETE INSTITUTE

INTRODUCTION

This paper summarizes certain data on the effect of vibration in placing concrete, obtained during the last year in the Materials Testing Laboratory at The University of Wisconsin. The conduct of this experimental work was made possible by a grant of funds from The Wisconsin Alumni Research Foundation. Acknowledgment is made to Messrs, C. E. Johnson, R. H. Krone, and B. H. Randolph, senior civil engineering students, for services rendered in conduct of these experiments, and to the Electric Tamper & Equipment Co., Ludington, Mich., for the loan of the vibrator.

The belief that the quality of present day concrete can be much improved by using drier consistencies, lower sand contents, and a vibrator in placing, led to these tests. Much assistance was obtained in making a selection of mixes from the enlightening paper, "Vibrated Concrete," by T. C. Powers[1], and most of the proportions described herein are similar to certain mixes reported by Powers.

One hundred eighty eight 6 by 12-in. cylinders were made and subjected to one or more of the following tests: density, absorption, freezing and thawing, and compression. Permeability tests under a 100-ft. head of water were run on 35 cylinders 9½ in. in diameter and 6 in. thick. Tests were also made on the effects of amplitude, speed, and time of vibration.

MATERIALS

Cement—The cement used in most of these experiments had a fineness of 93 per cent passing a 200-mesh, and in 1:3 standard sand mix exhibited tensile strengths of 295 and 358 p.s.i. at 7 and 28 days, respectively. The cement used in the tests on frequency of vibration had a fineness of 91 per cent and gave 7- and 28-day strengths of 318 and 417 p.s.i., respectively. Both cements required 23.5 per cent of water for normal consistency.

*Presented at the 31st Annual Convention, American Concrete Institute, New York, Feb. 19-21, 1935, in the author's absence.

†Professor of Mechanics, The University of Wisconsin, Madison, Wis.

[1]Journal Amer. Concrete Inst., June 1933; *Proceedings* Vol. 29, p. 373.

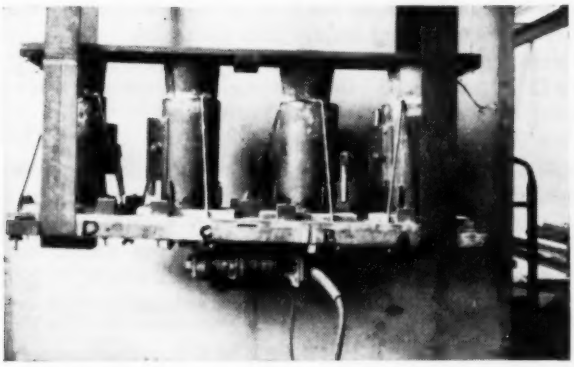


FIG. 1—ARRANGEMENT OF VIBRATOR, PLANK, SUSPENDING BELTS, AND FUNNELS FOR MOLDING 6 BY 12 IN. CYLINDERS

Aggregate—Janesville sand passing a ¼-in. screen and having a fineness modulus of 2.75 was used. Its rodded specific weight when dry was 111 lb. per cu. ft., its specific gravity was 2.70, and its absorption 0.6 per cent by weight.

Janesville gravel, the coarse aggregate, was screened and graded by weight as follows: 1 part ¼ to ⅜ in., 2 parts ⅜ to ¾ in., and 3 parts ¾ to 1½ in. in diameter. The fineness modulus of the mixed gravel in these proportions was 7.3; its rodded weight per cu. ft. was 105 lb.; its absorption 1.36 per cent, by weight, and its specific gravity was 2.68.

MAKING SPECIMENS

Mixing—The mixes used in the freezing and thawing, absorption, and strength tests and their principal properties are listed in Table 1. In the permeability tests the proportions were the same and the properties essentially the same. The mixes used in the tests on frequency and amplitude of vibration were similar to mix G of Table 1. Quantities were weighed and proportions are expressed in terms of weight.

Mixing was done by hand in metal trays and specimens were made and vibrated in batches of four. Air dry aggregates were used and allowance was made for absorption in proportioning the water and by allowing the mixed batches to stand for ½ hr. under damp canvas prior to molding. A minimum of water was used in all batches which were to be vibrated. After mixing, the batches had the appearance of damp earth but they exhibited no moisture or water gain after standing in the tray.

All of these mixes would be termed undersanded for poured or puddled concrete. Batches of the N-mix, 1:3½:6½, by weight, were tempered with enough water to give a 2-in. slump and the specimens were rodded in the standard manner.

Vibrator—The external vibrator was bolted to the under side of a 2½ by 14 by 53 in. plank and suspended by four 5½ ft. strands of 4 by ¼ in. leather belt as shown in Fig. 1. This vibrator consists of a small induction motor having a rotor equipped with two unbalanced weights. These weights can be adjusted to secure 6 variations in amplitude of vibration. The vibrator is designed to run at 3600 r.p.m.

Concrete was shoveled into the molds through the tar-paper funnels, shown above the 6 by 12-in. cylinder molds in Fig. 1, until level with the tops of the funnels. The funnels were removed after initial consolidation of the concrete had taken place.

Care was taken in making both the 6 by 12-in. cylinders and the 9½ by 6-in. permeability cylinders to prevent leakage from the molds.

Specimens were cured under wet canvas in the molds for 24 hours, then in a moist room at 68° to 72° F. until tested. Before storage, each specimen was marked according to the mix and position on the vibrator plank.

TESTING

Specific weight tests were made on all specimens when they were removed from the molds by weighing in air and under water. From these data and the weights of the batches, cement contents, solidity ratios, and voids were calculated (see Table 1).

Absorption tests were run on three 6 by 12-in. cylinders of each mix A to N after 28 days of moist curing. Specimens were heated at 212 to 220° F. for 5 hours, then immersed in water at room temperature and weighed under water to 0.01 lb. at ½ hr., 3 hr., and 48 hr. intervals.

Permeability tests were performed on the 9½ by 6 in. cylinders after they had been moist cured 28 days. The apparatus was the same as that used by Norton and Pletta[2] and G. W. Washa.[3] The average flow of water into the concrete for the interval from the 40th to the 50th hour of the test is the criterion used to measure permeability. Notes on damp spots, beads of water, and leakage from the free surfaces of the specimens were recorded when they occurred.

Freezing and thawing tests—After curing 28 days in the moist room, three cylinders of each mix B, E, and H, listed in Table 1, were subjected to 50 cycles, and three of each of the remaining mixes were subjected to 150 cycles of freezing and thawing. These cylinders were soaked in water at 70° F. for two days prior to the first freezing. They were then placed on end in open trays filled with water to a depth of one inch. These trays were conveyed on trucks into an automatic refrigerator. Readings on resistance thermometers in the centers of specimens indicate that the specimens were cooled to 32° F. in approximately 2½ hours and to 0° F. in 11 hours. The minimum temperature in the cold room during each cycle was —8° F. After 16 hours in the refrigerator, specimens were removed and thawed in running water at a temperature between 65 and 75° F. Thermometer records show that the specimens were thawed in less than 1-hour. At the conclusion of the freezing and thawing tests cylinders were soaked in water for one or two days prior to testing in compression.

TABLE 1—PROPERTIES OF MIXES USED IN STRENGTH, FREEZING AND THAWING AND ABSORPTION TESTS

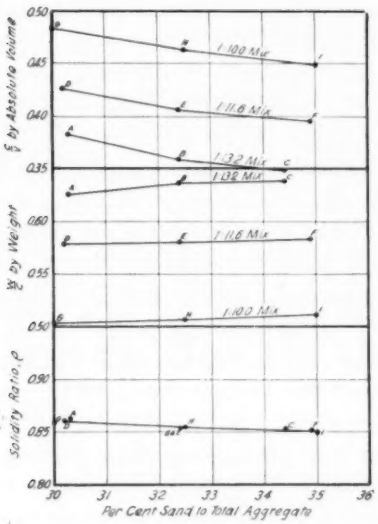
Each batch of four specimens was vibrated 1 minute at a speed of 3600 r.p.m. with machine running at ⅔ maximum amplitude. Twelve specimens were made for each proportion listed.

Mark	Proportions by Weight	Percentage of Sand in Aggregate by Weight	Specific Weight, Lb. per cu. ft.	Cement Content		Solidity Ratio, p	Cement Void Ratio, c/v	Water-Cement Ratio by Weight, w/c
				Sacks / cu. yd.	by Absolute Volume, c			
A	1:4.0:9.2	30.3	154.0	2.96	0.0528	0.862	0.384	.631
B	1:4.3:8.9	32.4	152.6	2.93	0.0523	0.854	0.359	.636
C	1:4.6:8.75	34.4	152.2	2.89	0.0515	0.852	0.350	.638
D	1:3.5:8.1	30.2	154.3	3.33	0.0595	0.861	0.427	.578
E	1:3.75:7.85	32.4	153.2	3.31	0.0591	0.854	0.406	.580
F	1:4.05:7.55	34.9	152.5	3.29	0.0588	0.852	0.396	.582
G	1:3.0:7.0	30.0	154.0	3.81	0.0680	0.859	0.483	.502
H	1:3.25:6.75	32.5	153.2	3.79	0.0676	0.854	0.464	.506
I	1:3.5:6.5	35.0	152.8	3.78	0.0674	0.852	0.456	.510
N	1:3.5:6.5*	35.0	151.4	3.66	0.0653	0.826	0.375	.768

*Rodded in placing, slump 2 in.

[2] "The Permeability of Gravel Concrete," JOURNAL Amer. Concrete Inst., May 1931; *Proceedings* Vol. 27, p. 1093.

[3] "The Efficiency of Surface Treatments on the Permeability of Concrete," JOURNAL Amer. Concrete Inst., Sept.-Oct., 1933; *Proceedings* Vol. 30, p. 1.



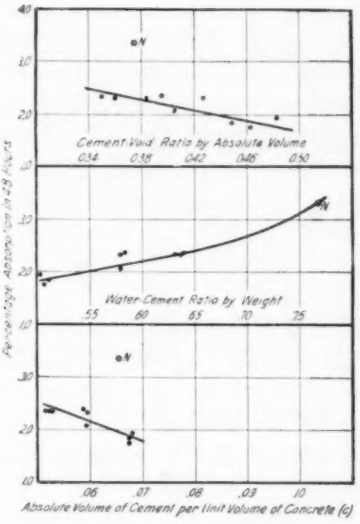

Cement Void Ratio by Absolute Volume
Water-Cement Ratio by Weight
Absolute Volume of Cement per Unit Volume of Concrete (c)

FIG. 2—EFFECT OF SAND CONTENT ON C/V, W/C, AND P
FIG. 3—RELATION OF ABSORPTION AT 48 HOURS TO C/V, W/C, AND C

Compressive strength tests—One day prior to testing for strength, specimens were capped on both ends with a mixture of neat Incor cement paste tempered with water containing 2 per cent of calcium chloride. Strength tests were made in a 3-screw Riehle gear-driven testing machine under a free speed of head of 0.06 in. per min.

Tests on amplitude and frequency of vibration—The speed of the motor in these tests was raised by increasing the frequency and voltage of the current. The amplitude was changed by adjusting the eccentric discs on the rotor of the vibrator motor. The power input was read on a wattmeter. Measurements of the amplitude of the vibration at the center of the plank in a vertical direction were made by means of a Cambridge vibrometer.

RESULTS OF TESTS

Comparisons of physical properties of mixes—It should be borne in mind that the vibrated mixes were all made with very dry consistencies which would exhibit no slump and that the N-mix had a 2-in. slump. From Tables 1 and 3 it appears that the solidity ratios and specific weights of the vibrated specimens are high and much higher than those of the rodded mix N. Furthermore, it will be noted that the densities of the mixes vibrated at high speeds were greater than those vibrated at low speeds. Although the data show that the solidity ratio increases with the time of vibration, part of that increase is due to a slight but perceptible splashing of water, fine cement and sand, from the tops of the cylinder molds which usually began at 30 to 45 seconds after starting the vibrator. Further comparing

TABLE 2—SUMMARY OF RESULTS OF STRENGTH, FREEZING AND THAWING, ABSORPTION AND PERMEABILITY TESTS OF VIBRATED MIXES

Each result represents three or four tests.

Mark	Proportions by Weight	Average Compressive Strength in Pounds per Square Inch—After				Percentage of Absorption by Weight—After			Average Flow for 40-50 Hr. Interval in Gal. per Sq. Ft. per Hr.
		28 Da. in Moist Room	50 Cycles of Freezing and Thawing	8 Mo. in Moist Room	150 Cycles of Freezing and Thawing	½ Hr.	3 Hr.	48 Hr.	
A	1:4.0:9.2	2950	....	3755	3565	1.01	1.71	2.33	.00070
B	1:4.3:8.9	2870	3355	....	....	1.10	1.69	2.32	.00118
C	1:4.6:8.75	2880	....	3675	3435	1.06	1.72	2.34	.00210
D	1:3.5:8.1	3325	....	4495	4320	1.15	1.75	2.31	.00048
E	1:3.75:7.85	3130	3520	....	....	1.01	1.55	2.08	.00042
F	1:4.05:7.55	3195	....	4395	3990	1.06	1.69	2.37	.00026
G	1:3.0:7.0	4390	....	5740	5200	0.97	1.44	1.95	.00020
H	1:3.25:6.75	4255	4635	....	....	0.90	1.30	1.78	.00018
I	1:3.5:6.5	4290	....	5475	5040	0.92	1.37	1.85	.00050
N*	1:3.5:6.5	2515	....	3205	2355	1.62	2.65	3.36	.00386

*Rodded in placing—slump 2 inches.

TABLE 3—EFFECT OF AMPLITUDE, FREQUENCY, AND TIME OF VIBRATION ON DENSITY AND STRENGTH OF CONCRETE OF 1:3:7 PROPORTIONS BY WEIGHT

Each value of strength, p, and specific weight represents 4 specimens.

Amplitude		Frequency r.p.m.	Time Vibrated, Sec.	Average Power Consumed Watts	Solidity Ratio, p	Specific Weight, Lb. per Cu. Ft.	Average Compressive Strength at 28 Days, p.s.i.
Setting	Actual,* Inches						
⅛	.015	3660	60	130	0.860**	154.3**	3525**
			120		0.866	155.1	3385
¾	.024	3470	60	230	0.862	154.3	3445
Max.	.028	3610	60	295	0.867	155.2	3610
			120		0.871	156.0	3530
				Average	.865	155.0	3500
⅛	.026	6330	30	480	0.863	154.9	3750
			60		0.866	155.3	3460
¾	.029	5240	25	1010	0.867	155.3	3605
			60		0.872	156.2	3360
Max.	.034	5000	20	1200	0.870	155.9	3590
			40		0.872	156.4	3575
				Average	.868	155.7	3555
⅛	.016	6900	20	1530	0.867	155.6	3670
			60		0.871	156.2	3650
¾	.020	6240	20	2470	0.869	155.5	3725
			30		0.869	155.9	3740
			40		0.873†	156.4†	3640†
				Average	.870	155.9	3685

*Average displacement from the mean position.
**One cylinder with stone pockets excluded—only 3 represented in average.
†Represents 8 tests.

mix N with the vibrated mixes shows that it had approximately the same cement content as mixes G, H, and I but a lower c/v-ratio and higher w/c-ratio.

The effects of the variations in sand content on the properties of the vibrated mixes are shown in Table 1 and Fig. 2. Each point in Fig. 2 represents the average for 12 specimens consolidated by vibration. From these data and within the limits of these tests, it appears

that the solidity ratio and cement-void ratio increased slightly, and the water-cement ratio decreased slightly as the sand content in the mix decreased. Pockets were noted near the bottom of one specimen of mix C and one of mix B. In the latter specimen the pockets exerted a material weakening effect. Whereas these mixes containing 30 and 32 per cent of sand were harsh working, they were, except as noted, readily vibrated into place. Small air pockets $\frac{1}{16}$ to $\frac{1}{4}$-in. in diameter found here and there over the surface of the vibrated specimen constituted the principal interruptions to the smoothness of the surfaces of most of the vibrated specimens.

Absorbtion of vibrated mixes—From the data in Table 2 and Fig. 3, it will be observed that the absorptions of all vibrated mixes were very low. The absorption of these mixes decreased with an increase in cement content and with an increase in the cement-void ratio, but it increased with the water-cement ratio. Both the vibrated mixes and the hand-rodded mix N conform to the absorption-water-cement relationship shown in Fig. 3, but the curves for the vibrated specimens between absorption and cement content and between absorption and the cement-voids ratio lie considerably below the points representing mix N.

Effect of vibration on strength and permeability—The data in Fig. 4 and Table 2 show that the strength-water-cement ratio relation for the vibrated mixes is similar to that for rodded mixes, but the strengths indicated for a given water-ratio is somewhat lower than for the corresponding curves for mix N. With the exception of data for mix C, the flows of the vibrated and N mixes all closely approximate the relationship between flow and water-cement ratio indicated by Norton and Pletta* for rodded concrete with 2- to 4-in. slump.

Fig. 5 indicates that for a given cement-void ratio the water tightness of a vibrated mix is superior to that of hand rodded mixes of wetter consistencies. Powers* states that the same strength-cement-void relation holds for both vibrated and rodded mixes. In these tests the rodded mix N fell below the strength-cement-void relation for vibrated mixes.

The data in Fig. 6 demonstrate that the compressive strengths of these vibrated mixes of dry consistency were markedly superior to the strengths of hand-rodded mixes of like cement content, also that for a given flow the vibrated mixes had much lower cement contents than those of wetter consistency placed by rodding.

In the permeability tests none of the vibrated specimens of 1:10 proportions, by weight, mixes G, H, and I, showed discolored or damp

*See previous footnote references.

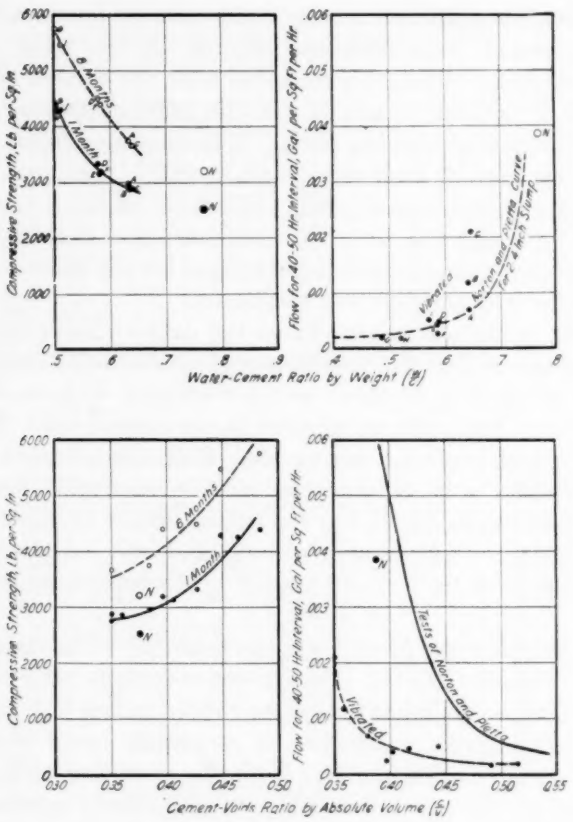


FIG. 4—RELATION OF STRENGTH AND FLOW TO WATER-CEMENT RATIO

FIG. 5—RELATION OF COMPRESSIVE STRENGTH AND FLOW TO CEMENT-VOIDS RATIO

surfaces, and all had low rates of flow into the concrete for the 40 to 50 hour interval. Hence for the 40-p.s.i. pressure used in these tests they can be considered practically water tight. Four of the specimens of the leaner vibrated mixes exhibited one or two beads of moisture or small damp spots on their surfaces. None leaked like the specimens of the N-mix.

Effect of freezing and thawing on vibrated mixes—Specimens from mixes B, E, and H after 50 cycles of freezing and thawing were tested for compressive strength. Since they did not show material weakening due to this number of cycles (see Fig. 9) breaking of the other specimens was deferred until they had been subjected to 150 cycles.

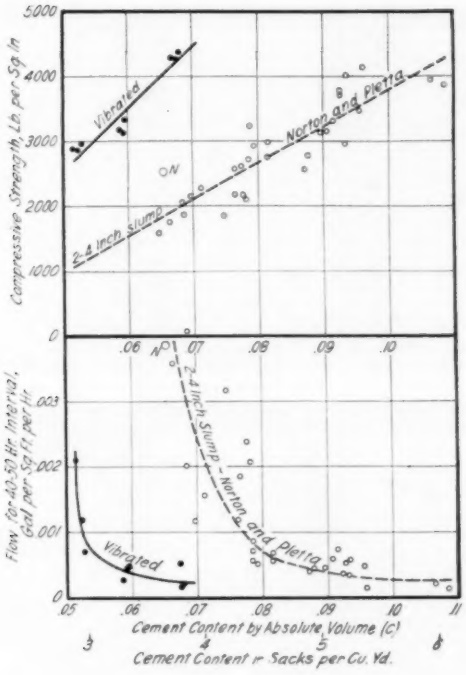


FIG. 6—RELATION OF CEMENT CONTENT TO STRENGTH AND FLOW

After 150 cycles of freezing and thawing the ends of the vibrated cylinders which were immersed during freezing and which are uppermost in Fig. 7(a) exhibited chipped edges. There were also occasional small surface spalls, as indicated by the arrow tips in Fig. 7(a), which appeared to be due to the break-down of weak pebbles adjacent to the surface of the cylinders. The immersed ends of the hand-rodded cylinders, marked N, Fig. 7(b), were badly disintegrated. The end of cylinder N12 crumbled away under the fingers and was removed before capping for test.

There is no question that the process of capping materially increased the crushing strength of the hand-rodded specimens which had been frozen and thawed. Hence the lower point marked N in Fig. 8 is higher than it should be with respect to the points representing the frozen and thawed cylinders molded by vibration.

The data in Fig. 8 and 9 show that these dense mixes of low cement content and dry consistency when placed by vibration possessed remarkably high resistance to freezing and thawing and were much

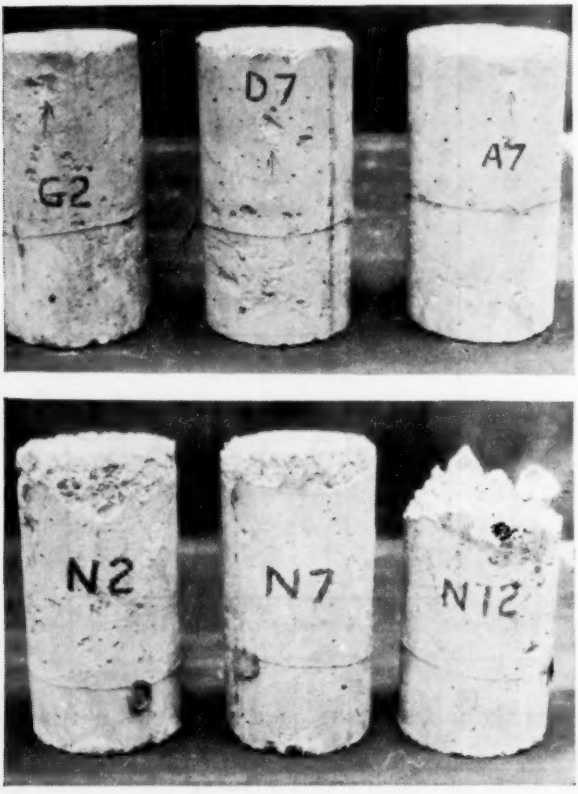


FIG. 7—EFFECTS OF 150 CYCLES OF FREEZING AND THAWING ON VIBRATED AND HAND-RODDED CONCRETE CYLINDERS. THE IMMERSED ENDS OF THE CYLINDERS ARE UPPERMOST IN THE ILLUSTRATIONS
(a—above) Vibrated Cylinders of 1:3:7, 1:3.5:8.1 and 1:4.0:9.2 mixes
(b—below) Hand Rodded Cylinders of 1:3.5:6.5 Mix and 2-Inch Slump

superior to the 2-in. slump hand-rodded concrete containing as much cement as the richest of the vibrated mixes.

The slopes of the lines in the diagrams in Fig. 9 show that the vibrated mixes A, B, C, with the lowest cement content, 1 part cement to 13.2 parts of aggregate by weight, though not as strong as the 1:11.6 or 1:10 mixes, exhibited a higher percentage of the 8 months normal strength after the freezing and thawing test. The data in Table 2 and Fig. 8 show that these 1:13.2 mixes were superior in strength before and after freezing to the 2-in. slump hand-rodded N-mix of 1:10 proportions.

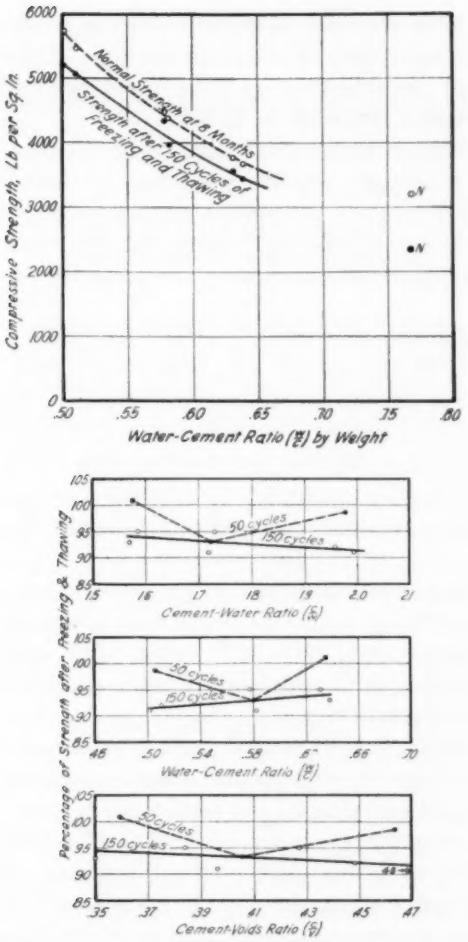

Compressive Strength, Lb per Sq In
Normal Strength at 6 Months
Strength after 150 Cycles of Freezing and Thawing
Water-Cement Ratio (W/C) by Weight
Percentage of Strength after Freezing & Thawing
50 cycles
150 cycles
Cement-Water Ratio (C/W)
Water-Cement Ratio (W/C)
Cement-Voids Ratio (C/V)

FIG. 8—EFFECT OF FREEZING AND THAWING ON THE COMPRESSIVE STRENGTHS OF VIBRATED MIXES AND RODDED MIX (N)

FIG. 9—EFFECT OF FREEZING AND THAWING ON COMPRESSIVE STRENGTH OF CONCRETE PLACED BY VIBRATION

Effect of amplitude, frequency and time of vibration—To ascertain the influence of speed, amplitude, and time of vibration 68 6- by 12-in. cylinders were made with the variables indicated in Table 3. Although these data are meager they show that within the limits studied, the magnitude of the amplitude is of less importance than the speed and length of time vibrated. The results indicate that the strength and density are slightly improved by using speeds of 5000 to 6500 r.p.m. By using speeds of 5000 r.p.m. and higher it was possible to cut down

materially the time required to consolidate the concrete as measured visually by the appearance of water on the tops of the specimens during molding and as indicated by subsequent determinations on the density and strength recorded in Table 3.

With the machine running at the normal speed of 3500-3600 r.p.m. and the maximum amplitude setting, a vibration period of one minute gave best results. At speeds approximating 6500 r.p.m., the actual amplitude for $\frac{1}{3}$ and $\frac{2}{3}$ setting was less than at 5000 r.p.m. but the time required to consolidate the concrete properly was only 20 seconds.

The power consumed increased both with the magnitude of the amplitude and with the speed. The power consumed at 6200 r.p.m. and $\frac{2}{3}$ maximum amplitude was over ten times that used at 3500 r.p.m. with the same setting for amplitude. In considering this ratio, it should be borne in mind that the machine was not designed for such operation, that its efficiency under such speeds was doubtless very low, and its life, if thus overloaded continuously, would be short.

CONCLUSIONS

1. By the use of a vibrator and a dry consistency, it is possible to make concrete carrying only 7 or 8 per cent of cement which will be dense, low in absorption, relatively water tight, highly resistant to freezing and thawing, and which will have a compressive strength at one month of 3000 to 3500 p.s.i.

2. In these tests vibrated concrete of 1 part cement to 13.2 parts of aggregate was superior in all respects to hand-puddled concrete having a 2-inch slump and carrying 25 per cent more cement.

3. Vibrated concrete cylinders of 1:10, 1:11.6, and 1:13.2 proportions, by weight, withstood 150 cycles of freezing and thawing and exhibited compressive strengths of 91 to 95 per cent of that developed by corresponding specimens moist cured over the same time interval.

4. Vibrated specimens of 1:10 proportions by weight showed no evidences of leakage on the free surfaces while subjected to a water pressure of 40 p.s.i.

5. From the tests made it seems probable that better consolidation of the concrete can be secured in shorter periods by using higher frequencies of vibrations.

6. By the proper use of a vibrator in placing concrete it appears possible to utilize drier consistencies, lower cement contents, and coarser gradings and to secure denser, more-volume-constant, water tight, and durable concrete than has been made with pouring or puddling methods of placement.

Vibration of Concrete on San Francisco-Oakland Bay Bridge

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Because most of the concrete in the San Francisco-Oakland Bay Bridge is exposed to sea water action, it was deemed essential to provide for concrete fabrication methods which would insure the greatest practical density in all units of the structure.

INTERNAL TYPE VIBRATORS

Preliminary to drafting the specifications a study of the internal methods of vibrating concrete was made by the Materials and Research Department of the California Division of Highways. The study consisted of an investigation of the efficiency of internal vibration by means of a high frequency vibration unit inserted into the concrete mass at the time of deposit. The internal vibrating unit tested was the Viber tamper type in which the vibration is produced by means of eccentric weights rotating on ball bearings inside a vibrator tube. The frequency of vibration was approximately 5000 r.p.m.

To observe the working of this type of tool, two units—one of the flexible shaft type and one of the rigid shaft type—were tried out on a comparatively stiff pavement concrete as it was dumped from the pavement bucket in a pile on the subgrade. This method was found inefficient when operating in a stiff concrete mass on a rigid base without any form work to provide a kickback to the vibrations. The operation, while ultimately effective, is slow for pavement construction.

We next constructed a special form to put the vibrator to a severe test in depositing concrete through reinforcement and around corners where the vibrating unit itself could not be brought within less than two feet of the extreme face. Fig. 1 shows a cross section of the form and reinforcement.

Two batches of 6-sack concrete were placed; one having a water cement ratio of .72 and a slump of 1¾-in., and the other a water cement ratio of .66 and a slump of less than ¾-in.

*Materials and Research Engineer, California Division of Highways, Sacramento.

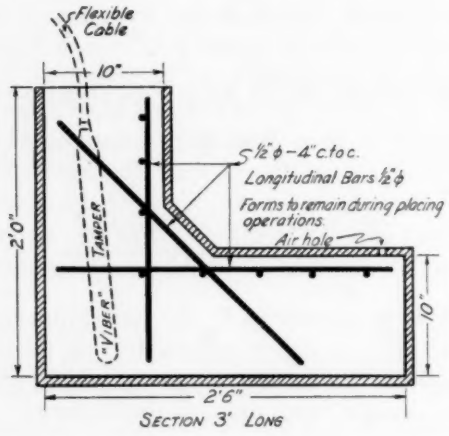

FIG. 1—CROSS SECTION OF FORM USED IN TESTING THE EFFICIENCY OF PLACING CONCRETE IN CONFINED AREAS BY MEANS OF INTERNAL VIBRATORS AND COMPLETE ABSENCE OF HAND TAMPING OR SPADING

The first batch was readily vibrated into place within a few seconds after depositing. The second batch, having a slump of but ¾ in. and requiring considerable spading to get out of the wheelbarrows, required more vibration than the wetter mix, but the ultimate result was entirely satisfactory.

This test demonstrated that a relatively stiff concrete could be readily distributed through a reinforcing mat for some distance from the face by means of the internal method of vibration, with not only a uniform distribution of aggregate throughout the mass but also without excess mortar on the faces, the only disturbing factor being the formation of air or water pockets adjacent to the forms.

The test indicated that it is perfectly practicable to use concrete of a much stiffer consistency than is usual in reinforced structural work.

As a result of these tests and other available data, a paragraph was inserted in the specifications for the Bay Bridge reading as follows:

The concrete after having been placed in the forms shall be thoroughly compacted by means of an approved internal vibrator. The vibrator shall be placed directly in the forms and shall not be attached to either the inside or outside of the forms, nor shall it be attached to the reinforcing steel in any manner.

The compaction shall be produced by inserting the machine in the concrete directly for a period of 20 to 30 seconds. This insertion shall occur approximately every 2½ ft., or where necessary to produce concrete of maximum density. Continuous attention shall be given to the working of concrete next to forms and around the reinforcing steel and embedded fixtures or inserts.

The above requirement for placement of concrete has made it possible to use concrete of a drier consistency on the structural portions of the Bay bridge work than is customary in this type of construction.

Most of the Bay bridge concrete is of Class "B" or 5 sacks of cement per cu. yd. The specifications provide a maximum water-cement ratio of .85, and a minimum strength at 28 days of 2500 p.s.i. More than 500,000 cu. yd. of such concrete have been placed to date on the five contracts under way, with an average strength of more than 3500 p.s.i. and a range of less than 5 per cent in average strength between the different contracts.

Two types of vibrators have been used, a small electric powered type for small units and confined places and a larger air powered type for larger masses.

Bridge Builders, Inc., Transbay Construction Co., and Healy-Tibbitts Construction Co. have used Model G Vibers. This is a rigid type vibrator, is pneumatic, with vibrator tube connected directly to the motor. The motor develops about 2½ h.p. and is of the rotary vane type. At 80 pounds pressure approximately 55 cu. ft. of free air per minute are consumed. The vibrator tube is 30 in. long and 3 in. in diameter and weight of vibrator about 57 lb.

Maximum speed developed with this vibrator will vary with the pressure of air, but with the air pressures used on the bridge project will according to the manufacturer of the equipment, run from 8,000 to 10,000 r.p.m.

The first of these models furnished for use on the Bay bridge were among the first built by the manufacturers. They differ very little from those now built. Most of the changes are in strengthened parts (where such changes were found necessary) the use of special steels, heat-treating and machining parts to very close tolerances.

Healy-Tibbitts Construction Co. used one Model K pneumatic Viber with motor identical with model G. It is equipped with a 12-foot flexible shaft to the end of which is attached a vibrator tube 2¾ in. in diameter and 20 in. long.

Under similar conditions, the speed of Model K will be somewhat less than that of model G although (according to the manufacturer) it should develop well over 8000 r.p.m. Difference in speed is due to loss of power through the flexible shaft.

The Clinton Construction Co. used Model 10 electric Vibers exclusively on both contract 5 and 8. It has a 12-ft. flexible shaft and 2¾-in. tube similar to those of Model K. The vibrator tube weighs 25 lb.

The motor is a 1⅓ h.p. 110-volt universal, weighing 47 lb. The armature turns at a speed in excess of 10,000 r.p.m. Motor is geared to deliver 4500 to 4750 r.p.m. at the vibrator tube, under load. Because of its lower speed this equipment does less work.

On Yerba Buena Island, T. E. Connolly, subcontractor under the Clinton Construction Co., used one Model G Viber and two Model 20 Vibers. Model No. 20 differs chiefly from Model K in having a 3-speed throttle which will give the operator instant control of speed of vibration varying from about 5000 to over 8000 r.p.m. There are two distinct advantages to this: Slower speeds can be used where relatively thin sections are to be vibrated and high speeds can be used where mass work is to be done or where extremely dry concrete is to be compacted; with the 3-speed throttle the starting torque on the flexible shaft core is less, due to the more gradual starting of the motor.

As a general thing, the electric models are best suited for building construction and pneumatic models on heavy engineering projects.

The volume which can be effectively handled per vibrator depends naturally upon the type of vibrator and the class of concrete.

The specifications around the smaller electric type unit require that the contractor shall furnish and use one vibrator for every 25 cu. yds. of concrete placed per hour. Vibrators on the job are handling from 25 to 40 cu. yd. per hour of mass concrete placed in deep layers. This larger output is handled with the large air driven vibrators. The maintenance of the equipment is high, couplings and shaft probably requiring the most frequent replacement. Electrical connections are often neglected and, therefore, very often cause delay. The air type motors frequently freeze.

We found that ⅝" ply wood and a tight mixture will hold on an 8 ft. lift. It was found necessary however, to decrease the spacing of studs in some cases to 14 in. and the wales to 3 ft. and to increase the ties approximately one third. Tightness of forms was found more essential with the internal vibration than with the ordinary hand placement methods.

It was found possible to handle harsh mixes more satisfactorily with the internal vibration method of placement and, therefore, if harsh mixes must be used on account of lack of availability of materials properly graded for a plastic mix, the internal vibrator is an assurance of more uniform fabrication.

Slumps have been satisfactorily used on the work ranging from 1 in to 4 in.

The extent to which the mortar proportion could be changed was found to depend on the type of the work. In large mass construction, the mortar may be kept at a minimum; whereas, in thin forms full of steel it may not be found practicable to reduce the mortar content below the standard for hand placement methods.

The concrete should be deposited in layers although appreciably thicker layers can be handled to satisfaction with vibrators than can be handled with hand tamping.

The distance from the forms with which the vibrators can be satisfactorily handled depends on the mix. In the wetter mixes, the vibrators can be placed closer to the forms than with the dry stiff mixes. Segregation may occur if the mix is too loose, therefore an additional argument for the use of tight mixes. If the mix is plastic, however, it does not tend to spread the aggregate, but does tend to draw mortar toward the vibrator so that when the vibrator is withdrawn, a core of mortar is likely to be left.

In general, the favorable effects of vibration may be stated to be greater capacity in confined areas, more effective consolidation, better surface, the ability to place drier concrete and to work the concrete around the steel to better advantage than is possible with ordinary hand placing methods.

The cost for labor is somewhat less with the use of the vibrator than without. There is an increase in the cost of the forms. This increase is more than absorbed by the saving in other items of cost, including saving in cost of patching unsatisfactory surfaces. Very little surface patching has been found necessary on the Bay bridge units. Where rock pockets have occurred in the surface, it has been due either to faulty use or entire lack of use of the vibrator in the defective areas.

PAVEMENT TYPE VIBRATORS

The specifications for paving the roadway decks provide that:

> The roadway slabs shall be consolidated and compacted by means of high frequency vibration applied to its upper surface to expel entrapped air and surplus water, to eliminate all porosity and simultaneously bring the concrete into intimate contact with all embedded reinforcement.

In conformity with these specifications the contractor is using two Jaeger-Lakewood automatic finishers with Jackson vibrator attachment; one finisher being approximately 30 ft. long to span the full width of the lower roadway, and the other approximately 20 ft. long to span each one of the three lanes of the upper roadway.

The pavement slab, varying from 6 in to 6½ in. thick, is heavily reinforced with trusses spaced 7½ to 8 in. centers (Fig. 2), the space

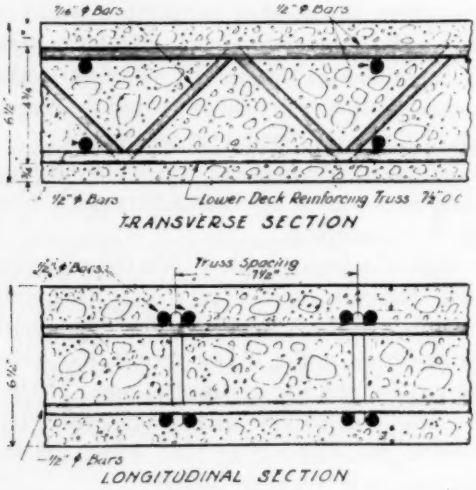

FIG. 2—DETAILS OF REINFORCED PAVEMENT SECTION, SUPERSTRUCTURE EAST BAY CROSSING SAN FRANCISCO-OAKLAND BAY BRIDGE

Lower roadway slab standard portland cement concrete 1½ maximum size aggregate Similar reinforcing in upper roadway deck of light weight concrete Trusses 4¼ high Spaced 8 in o c Slab thickness 6 inches Maximum size aggregate 1 in.

between the bottom of the truss and the bottom of the concrete slab is but ¾ in. The top and bottom tie rods further increase the cross-sectional area occupied by the reinforcing steel. The maximum size of aggregate is from 1½ in. to 1¾ in. for the lower roadway and 1 in. for the upper roadway which is of light weight aggregate.

Paving the roadway slabs has just been started, therefore, all of the problems in connection with the operation of this type of equipment and the proper design of the mix have not been entirely solved; however, 6 sack concrete with a water cement ratio of .72 and a slump of 2 in. or less is being satisfactorily placed with this type of vibrator. Other than for the first run where the concrete was insufficiently vibrated, there is complete lack of honeycomb on the underside of the slab. The compressive strengths at 10 days (the greatest age now available) average 3000 p.s.i. for the standard aggregate concrete.

Satisfactory results with this method of placement have likewise been secured on the upper roadway light weight aggregate concrete where 6.5 sack concrete with a water cement ratio of 0.60 and 10 day strengths of 2300 p.s.i. has been satisfactorily placed.

Vibration as an Aid in Placing Better Concrete

BY LEWIS H. TUTHILL*
MEMBER AMERICAN CONCRETE INSTITUTE

Over-emphasis of the water cement ratio has probably resulted in nearly as much poor concrete, due to trying to place it too dry, as has resulted from ignorance of this fundamental influence on concrete quality in permitting it to be placed much too wet. With the advent of vibration as an aid in placing concrete, there is considerable evidence that a good margin on the drier side of the ordinarily acceptable consistency may be obtained successfully without the discouraging results obtained by the over-zealous practitioners of the water cement ratio theory.

But again caution and good judgment are required, for the vibrator, whatever form is used, has its limitations. It must be remembered that after all it is merely a tool, that its noisy presence on the job falls a long way short of insuring a perfect job of concrete placing regardless of the mix, the grading, the water cement ratio, the manner in which the concrete is handled and placed, and last but by no means least, the manner in which the vibrator itself is used.

The great effectiveness of vibration as an aid in placing concrete lies in the fact that for the moment it transforms stiff or plastic masses of concrete or its mortar matrix into a heavy liquid which is strongly urged by gravity to seek and completely fill all hitherto unfilled spaces accessible to it. This is its only ability. Thus it may be seen that vibration must not be expected to reincorporate coarse aggregate carelessly and excessively separated by improper handling of the concrete; to fill voids in aggregate when there is barely enough mortar in the mix to do so; or to transmit itself effectively through a mass so dry that there is no cohesion of coarse aggregate and mortar, except as used in the concrete products industry.

Particular emphasis is placed upon the importance of proper handling and placing the concrete. If the full benefit of vibration is to be

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derived from the use of practical minimum water-cement and sand-rock ratios, its efficiency must not be impaired by using it to transport the concrete or remix it in the forms as well as to consolidate it. Transportation and mixing both may be accomplished by more effective and less expensive means.

In so far as possible, and it usually is possible, concrete should not be transported in the forms by causing it to flow laterally from one point to another. It should be put where it is to stay with the next unit beside it and so on along the work until a shallow lift is completed. Thus when the concrete is vibrated from within or without the effect is not largely dissipated in lateral motion, but is concentrated in vertical settlement and consolidation of the concrete, thereby making the operation wholly effective to the end desired.

With placing equipment arranged to place each small unit of concrete in its final position, it is usually possible, with a little extra precaution, to eliminate separation completely and avoid remixing. Much trouble in this respect would be avoided if plant and equipment designers and many construction men would cease thinking of concrete as a liquid but would consider it as it is: a mixture of a wide size range of solid particles held loosely together with water from which the larger particles will bound away at the slightest opportunity. Such opportunities are most frequently presented by discharging concrete at an angle less than 90 per cent from the horizontal into concrete in the forms or into recurring hoppers or buggies. The smaller this angle the greater the separation, the farther will the rock fly from the concrete. Vertical drop is the solution to this problem, first as applied to concrete dropping into place in the forms, then back through all handling operations to the mixer. Concrete should be confined at all points of change in direction of flow. In all cases flow of concrete into forms, hoppers or buggies should be directed into the fresh concrete and not against the sides. Sloping discharge and twin discharge hoppers should be replaced with hoppers having only one discharge and that arranged in the bottom so that the concrete will drop vertically into buggies, cars, or belts receiving it. Equipment filling such hoppers should be arranged so that concrete will fall into them vertically without separating. For handling expediently the relatively drier concretes which become practical with vibration, extra large hopper gates and throats to chutes and articulated down pipes are necessary. Twice the usual area is a safe precaution.

Such safeguards against separation are essential to any completely successful concrete placing operation, but if they are followed where

vibration is used, the exceptional benefits of this treatment may be concentrated in properly placing a concrete of minimum water-cement ratio with maximum efficiency.

Conditions or character of the job will assist in dictating the type of vibrator. The size and power of the unit should be as great as may be efficiently operated in the space available. Under-size or under-powered vibrators are undesirable as they give a false sense of accomplishing results which are not evident when the forms are removed. Whenever possible an internal vibrator should be used, even for mass and slab concrete. Considerable concrete of this type has been handled by surface vibration, but large internal vibrators have largely replaced them on mass work and it is believed that equipment men developing application of internal vibration to slab and paving concrete are working on the correct principle. Surface vibration of slabs brings the poorest part of the concrete and water to the top where usually the best quality is required. Vibrators of the type which may be clamped to the outside of the forms are useful when there can be no access to the concrete with internal vibrators. They should be attached so that the principal path of motion of the vibration is in a horizontal plane. Unfortunately this type of vibration seems to draw air bubbles to the surface which other types leave distributed in the mass.

Form vibrators should be spaced about six feet apart along the level of concrete being raised. This space will vary with the power of the vibrator and the rigidity of the form. Spacing too wide will cause over-vibration at the vibrators or under-vibration between them, or both. They should be moved up so that they are never more than two or three feet below the rising surface of the concrete. Properly powered and operated internal vibrators will take care of 10 to 20 cubic yards of concrete per hour depending upon its accessibility and how well it is placed.

In internal vibrators higher frequencies have seemed most effective but, on the flexible shaft machines, have been the most expensive to maintain. In the electric machines having lower frequencies, greater amplitude has been most effective. Upkeep has not increased in proportion to the increase in amplitude. Form vibrators have been found to be most effective at a high frequency and a small amplitude, thus imparting a general vibration to the mass as a whole rather than a succession of sharp blows to the adjacent surface.

Stronger forms are required for form vibration than when relatively drier concrete is placed and worked with an internal vibrator. No more than first class practice in form construction is required but

poorly built forms of undersize material are a hazard if vibration is used. The forms must be tight however or much good mortar will be drawn off through the cracks by vibration leaving unsightly streaks on the new work.

No liberties should be taken with mixes and proportions because the concrete is to be worked in the forms with a vibrator. Fine material smaller than $\frac{3}{8}$ in. should compose not less than 33 per cent to 40 per cent of the mixed aggregate depending on the shape of the aggregate particles, the well rounded aggregates requiring less of the fine material. The undersanding of mixes to be vibrated is attended by more difficulties in handling the concrete and in getting proper results than is warranted by the small economies or increases in strength or density which are made by so doing.

Water content may be reduced and water-cement ratios may be lowered because relatively drier concrete may be placed and consolidated in the work with vibration than without it. Using an effective type of internal vibrator in placing reinforced concrete, average slumps have been reduced in walls, columns and deep beams from 8 in. to 5 in. and in slab and beam floors and more accessible work from 6 in. to 3 in. This may result in stronger concrete for the same cement content, or the same strength for less cement.

Wet concrete should not be vibrated and no concrete should be vibrated any more than is necessary to settle it properly into place. 8-in. slump concrete vibrated in a 6 x 12-in. cylinder showed only poor quality mortar in the top 2 in. Excess vibration causes settlement of the coarse aggregate and concentrates fines and inferior material at the top of the concrete. Sufficient internal vibration is evidenced by the settled satisfactory appearance of the surface of the concrete when the vibrating unit has been placed deeply enough to penetrate a few inches into the previous lift. The best vibration is that which is applied in many places, short distances apart, but for only a few seconds in the concrete at each point. This is best done following quickly behind the concrete as placed ahead in a shallow horizontal lift of 15 or 20 in. so that the operations of placing and vibrating are not in conflict but in close sequence. Internal vibrators may be operated anywhere in the concrete where it may appear that they will be beneficial. In work in which the forms are as tight as they should be, there appeared to be no evidence that the vibrator should be kept away from the forms, except that wooden forms may be scarred and roughened by sand and gravel being thrust into the surface by vibrator. Steel forms are not so affected. For most construction this tendency will not be significant. Within forms, contact of the vibrator with the reinforcing is of distinct

advantage in more widely distributing the influence of the vibrator. Some fear has been expressed that on unformed reinforced sloping slabs the contact of vibrators with steel may cause slipping or slumping of the green slab below the placing operation. This has not been proven here for as yet no canal lining has been placed.

We have not been able to eliminate air bubbles on vertical surfaces and those where the forms are even slightly above the concrete. Where the forms are under the concrete such as above the spring line of an arch there is no difficulty. The number of these bubbles is not increased by internal vibration but it does not eliminate them. Where accessible, spading with a flat tool along the forms will reduce the number of these bubbles but will not remove all of them. Otherwise supplementary spading is not required if the concrete is properly placed without separation and the vibration is as efficiently done as it may be. Few bubbles have been found on surfaces where the concrete was wet or where the forms have had many joints or were porous. In this case there is little resistance to the escape of the entrapped air. But when the relatively drier concretes are placed against a well oiled steel form the resistance to the rise or escape of the bubbles is so great that a great many of them are still present when the form is removed.

The only damage or disadvantage from the use of vibration comes from excessive application of it, particularly in concretes of the wetter consistencies and especially when the wetter consistencies are used in slabs requiring finishing. Unless vibrated very sparingly, amounts of water and fines are brought to the top of such slabs which greatly impair the quality and durability of the finished surface.

The principal benefit of vibration is the improvement in quality or economy of the concrete in all properties which are direct functions of the water content, due to its ability to consolidate properly concrete of relatively drier consistencies. Tests have shown that internal vibrators actually increase the density of concrete over hand placing by about five-tenths per cent. This is probably largely due to the escape of entrapped air.

When the quality or economy of the concrete is considered vibration adds little if any cost to concrete work. Forms for vibration have to be no better than they should be otherwise. Labor if anything is reduced. There is no change in material costs unless cement is saved due to using relatively drier concrete. With proper vibration patching is eliminated and the first steps of the finishing operation are made easier.

It appears to us that the effective trend of vibration is toward the internal type. The greatest need is for the elimination or vast im-

provement of the flexible shaft, or the development of electric machines which can give equal performance with the motor submerged in the vibrator so that only a flexible handle and cord are required.

Byron Hill, a contemporary concrete engineer in this organization, has cooperated in the preparation of these comments by making available observations made by him on other work.

High Frequency Vibration As Applied to the Construction of Concrete Pavements

BY F. H. JACKSON*
MEMBER AMERICAN CONCRETE INSTITUTE

As the result of experiments at Arlington, Va., in 1929 and 1931, the Bureau of Public Roads has established quite definitely that the driest consistency which can safely be used in the construction of concrete pavements by the usual machine finishing methods corresponds to a slump in the neighborhood of 2½ in. These tests indicated that mixes drier than this, unless greatly oversanded, were almost invariably honeycombed to such an extent as to affect seriously the beam strength of the pavement, even though it was possible in many cases to secure a satisfactory surface finish. The fact that it is necessary to sacrifice the increased strength and density which should accompany the use of a drier mix in order to maintain workability leads naturally to the consideration of vibration as a possible means of making such mixtures workable under the conditions encountered in pavement work.

According to Bureau records, the first attempt to apply modern high frequency vibration to the placing of concrete pavements was in Ohio in the fall of 1930. A conventional type Lakewood finishing machine with electric vibrating units mounted directly on the screeds was used in this work. This was followed in 1931 by a project constructed in Texas using the same type of equipment. In this same year a pavement was constructed in New Jersey using a Baily-Davis vibrating roller, mounted on a frame attached to the rear of a standard finishing machine. In this type of equipment the vibrating element rested directly upon the concrete and did not come in contact with the side forms.

Since these initial demonstrations, a number of experiments have been conducted both by the Bureau and by the State Highway Departments of Illinois, Michigan, Missouri, New Jersey and Ohio. In 1931 the Bureau conducted an extensive series of tests using vibrating

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screeds similar to those employed in Ohio the year before.[1] These tests were conducted in such a way as to make it possible to evaluate directly the effects of vibration through the testing of beams taken from sections of the pavement. The efficiency of the particular finishing method employed was judged by strength and density tests as well as by determinations of the amount and distribution of honeycombing on the bottom of the slabs. It was found that, for the finishing equipment and material combinations employed, concrete having a slump as low as 1 in. could be satisfactorily placed, as compared to a minimum of 2½ in. for the usual methods. The reduction in slump made possible by vibration resulted in an average reduction in the water-cement ratio of about ½-gallon per sack of cement and an increased strength and density approximately comparable to this reduction. This indicates that the improvement in quality was due entirely to the fact that under vibration the drier mix became plastic and workable, thus extending the application of the water-cement ratio strength law to mixes of this consistency when vibration is used. This work also disclosed that under vibration as much as three-fourths parts of coarse aggregate in terms of cement could be added to a typical standard paving mix, thus increasing the density without sacrificing either strength or uniformity. Such a mix would contain approximately one-half sack of cement per cubic yard less than the unvibrated concrete. In this connection there were indications that the effects of vibration were somewhat more marked when crushed stone was used as coarse aggregate as compared to rounded gravel.

The work done in Illinois, Missouri, Michigan and Ohio has substantiated in general the conclusions reached by the Bureau as the result of its 1931 work. In Missouri[2] the experiments indicated the possibility of reducing cost without sacrificing quality. It was found from core tests that a vibrated mix containing approximately 10 per cent less cement than the standard mix had approximately 10 per cent greater strength and also that a 17 per cent reduction in cement resulted in only a 5 per cent reduction in strength. Core tests also showed that the vibrated concrete was somewhat more uniform than the unvibrated, which verifies the conclusions reached by the Bureau from its study of concrete beams taken from the pavement. The favorable results secured led Missouri to repeat this work last year.

[1] "Vibration and Delayed Finishing Improve Pavement Concrete," by F. H. Jackson and W. F. Kellermann. *Public Roads*, Oct. 1933, Vol. 14, No. 8, p. 129; *Proceedings*, American Society for Testing Materials, 1933; Abstracted in *Concrete*, Aug. 1933, Vol. 41, No. 8, p. 8.

[2] "Vibratory Finishing Machine for Concrete Pavements," by F. V. Reagel. JOURNAL Amer. Concrete Inst., June 1933, *Proceedings*, Vol. 29, p. 391.

No results of this study are as yet available. Michigan[3] concluded that the vibrating finishing machine was efficient in handling, compacting, and finishing dry, harsh mixtures. It was noted, however, that maximum efficiency could not be attained due to difficulties encountered in hand spreading and hand finishing operations. This point was brought out also in connection with the Missouri work, indicating that if maximum efficiency is to be secured through vibration, it will be necessary to revise present practice as regards forming joints, constructing lip curb, etc.

Illinois[3] noted the ability of vibrating finishing machines to handle and consolidate dry, harsh mixtures. In this work the most suitable consistencies for vibration were found to be about ½-in. slump used in a mix in which the amount of fine aggregate was 33 per cent of the total aggregate in the case of gravel mixtures and 35 per cent of the total aggregate in the case of crushed stone mixtures. These experiments indicated that a reduction in cement content of approximately one sack of cement per cubic yard of concrete could be obtained. This is considerably drier concrete with a correspondingly greater saving in cement than the Bureau found in its 1931 tests. In the Bureau work a minimum slump of about 1 in. and a saving in cement of about one-half sack were indicated.

Since 1931 practically all of the pavements constructed in New Jersey have been consolidated by vibration. R. B. Gage concludes[3] that through vibration the cement content can be reduced up to one-half sack per cubic yard without lowering the strength of the concrete. The chief advantage of vibration as a result of his experience is increased density through the use of a mix containing more coarse aggregate, resulting in less excess mortar on the surface.

Subsequent to the tests conducted by the Bureau in 1931, additional types of vibrating equipment have been developed. These include the Blaw-Knox vibrator, in which the vibrating member is suspended between the screeds of the finishing machine and does not rest directly upon the forms as is the case with the vibrating screed employed by Lakewood. In this respect the Blaw-Knox equipment operates in much the same way as the Baily-Davis roller previously referred to. These new developments in equipment together with the fact that the Lakewood vibrating screed type has been considerably modified by the use of heavier and wider screed members, resulted in the Bureau repeating its 1931 experiments last year, using all of the latest type equipment available. In these later tests, studies were made of the effect of vibration with the concrete proportions varied as follows:

[3] "Vibrated Concrete Pavements." *Engineering News-Record*, April 26, 1934, p. 528; and May 3, 1934, p. 561.

1. Maintaining the water-cement ratio constant and increasing the amount of coarse aggregate by uniform steps until the value of b/b_0 had been increased from 0.72, the base mix for unvibrated concrete, to 0.82. This automatically decreased the slump from 2½ in. for the base mix to around ¾ in. for the harshest combination. The cement factor was reduced from 6.0 sacks to about 5¼ sacks.

2. Maintaining the water-cement ratio constant and increasing the amount of both fine and coarse aggregate by uniform steps until the slump had been reduced to about ¾ in. This reduced the cement factor to about 5 sacks per cubic yard.

3. Maintaining the cement factor constant, and reducing the water-cement ratio and at the same time decreasing the amount of sand and increasing the amount of coarse aggregate.

The object of the first two series of tests was to investigate the economies which might be effected through the ability of the vibrators to handle drier concrete than the standard finishing machine. This was on the assumption that the strength could be maintained constant by maintaining the same water-cement ratio. The object of the third series was to determine the improvement in quality made possible through the use of drier mixes when the quantity of cement was maintained approximately constant. This investigation involved the placing of 368 sections of concrete, each 8 ft. long and 10 ft. wide, or the equivalent of a little more than one-half mile of pavement. The sections were all constructed during the summer of 1934 and will be tested during the coming summer.

The latest type Lakewood vibrating screed, the Blaw-Knox vibrator and several devices supplied by the Concrete Vibrator Equipment Co. (formerly Baily-Davis) were investigated. In general, the equipment was operated in accordance with the manufacturer's suggestions. Practice in this regard seems to have become fairly stabilized insofar as frequency of vibration is concerned, at 3,600 to 4,000 impulses per minute. At this frequency it has been calculated that the vibrators of the type used in our investigation delivered around 1,200 to 1,300 ft. lb. of energy. For certain sections this intensity was increased to about 1,500 ft. lb. by increasing the amplitude of vibration. In other cases the effect of the amount of vibration given at any point was investigated by operating the vibrator at an increased forward speed. No attempt was made to operate at frequencies higher than 4,000 because of the feeling that this frequency was about the highest which should be used with equipment as now designed.

It is impossible of course to draw any definite conclusions from the work at this time, except to state that, insofar as handling and finishing the pavement sections were concerned, satisfactory results were secured with concrete having a slump of about ¾ in., with 28 per cent of sand by weight of total aggregate where gravel was used as coarse aggregate, and 32 per cent of sand where crushed stone was used as coarse aggregate. For ordinary finishing these same materials required 33 per cent sand in the case of gravel and 38 per cent sand in the case of crushed stone. These observations apply to sand having a fineness modulus of about 3.0. With finer sands, somewhat lower sand contents could undoubtedly have been used. So far as it is possible to judge without an examination of the structure of the concrete, the limiting slump which can be handled appears to be about ¾ in., although in some cases mixes having as low as ½-in. slump were satisfactorily finished.

Although virtually everyone who has experimented with vibration in connection with the placing of pavement has reported favorably as to its possibilities, the method has not been tried to any extent in any state except New Jersey. There seems to be a marked hesitancy on the part of the states to depart from standard practice in this regard due probably to a feeling among engineers that the advantages so far indicated are not of sufficient magnitude to balance the increased equipment charge. It is true that, measured solely by strength, the improvement so far indicated is not great nor are the possibilities of economies through cement saving startling. Nevertheless, it must be remembered that all of our experiences with concrete in general, both in the laboratory and in the field, indicate that vibration probably imparts other desirable qualities which, although not so easily evaluated as strength, are fully as important. The Bureau tests indicate that the increased density and lower absorption which are theoretically possible through the reduction in the water-cement ratio and increase in the quantity of coarse aggregate are actually realized under vibration. If this is true, it follows as a matter of course that the durability of the concrete will be improved. As we all know, durability is of prime importance in pavement concrete and any possibilities which vibration may have in contributing to this end should be very thoroughly explored.

It is the writer's opinion that the equipment now available can be utilized to advantage to this end and that, if operated properly, the improvement in pavement quality will more than justify the slight additional equipment cost involved. As a definite suggestion to any who may care to specify vibration, he would suggest consideration of the following: A specification based on a maximum water-cement ratio (using the same value as with standard construction) but with a

provision that the proportions be adjusted to give approximately a 1-in. slump and that the minimum quantity of sand necessary to give the required workability be used. The proper percentage of sand will, of course, vary with conditions. It is as illogical to specify an arbitrary proportion without regard to the materials as it is when standard finishing methods are employed. It is also just as essential that the basic rules governing the making of good concrete be adhered to. The difference lies in our concept of what constitutes "workability." For this method of placing, *plasticity under vibration* is the critical measure of workability rather than the plasticity of the freshly mixed concrete.

Subject to the above general rule, the bulk of experience to date seems to indicate that when vibration is used the percentage of coarse aggregate may be increased about 5 per cent in terms of weight of total aggregate. In other words, mixtures which for ordinary placing would be designed for 35 per cent sand should, with similar materials, be proportioned on the basis of about 30 per cent sand.

There is an additional point which cannot be too strongly emphasized. The type of paving mix commonly used in pavement construction (that is, one carrying a rather large proportion of sand with a slump of 2 in. or more), is not only not improved by vibration but the concrete may be actually damaged due to segregation and the accumulation of an excess of sloppy mortar on the surface of the pavement, with consequent tendency to surface scaling and other evils. Oversanded and wet mixes must be avoided. On the other hand the principle of vibration if properly applied can, in the writer's opinion, be utilized to improve materially the quality of paving concrete.

A final word as to the future. If the maximum benefits are to be secured through vibration it is obvious that the present methods of handling the concrete from the mixer drum to the subgrade must be materially revised. Present equipment is designed to handle plastic mixtures. However, it is undoubtedly quite practical to design distributing equipment adapted to handle dry concrete and the equipment manufacturers will undoubtedly take care of this end of the problem if the engineers manifest a sufficient interest in the possibilities of vibration for improving the quality of the concrete. In this connection, the equipment manufacturers are to be commended for their progressive attitude in developing vibrating equipment to its present state in the face of the rather problematical attitude of the engineering profession as to its utilization.

A complete report giving the Bureau's findings in connection with the work now under way will be made available as shortly as possible after the completion of the testing next summer.

Mosaic Ceilings, U. S. Department of Justice Building*

BY JOHN J. EARLEY†

MEMBER AMERICAN CONCRETE INSTITUTE

This paper is the fulfillment of a promise made more than a year ago to Harvey Whipple. He visited Washington while we were doing the mosaic concrete ceilings of the new United States Department of Justice building, and insisted until I promised to write for publication in the Journal of the American Concrete Institute, an article pointing out some of the unusual aspects of the work[1].

I have been greatly attracted by the theme of construction which our studio developed for these ceilings. Its value to architectural concrete became immediately apparent and from it we have already developed a system of prefabrication for the walls of concrete houses. It opens vast possibilities for new development both in residential and monumental architecture.

I hope that this paper will stimulate discussion. At this time new ideas are needed, especially ideas for the application of well established principles to new problems. Such ideas may appear at almost any turn of a technical discussion and one such idea will fully repay the effort spent in developing it.

Zantzinger, Borie and Medary of Philadelphia, architects of the new Department of Justice building at Washington are noted for their ability, their careful study of detail and their knowledge of materials. They are always interested in new materials but particularly in new methods for the application of well established principles of craftsmanship. This interest prompted them to study mosaic concrete, which they subsequently accepted as an architectural medium fit for use in monumental structures.

*Received by the Secretary, American Concrete Institute, May 16, 1935.

†Architectural Sculptor, Washington, D. C.

[1]Mr. Earley's mind had gone on to other newer and more interesting adventures—but as he intimates, it might not have gone on to the principles underlying his small house project but for solutions of problems first presented to him in the Justice building. As a step in a technique the present paper is best read therefore before Mr. Earley's convention paper on house construction in this Journal. For this reason, to make the A. C. I. record of the Earley technique complete, the Secretary insisted—insofar as he might insist—Editor.

Because of their thorough knowledge of mosaic they understood readily that the tesserae of mosaic could be replaced by the particles of mosaic concrete, with great economy and without loss of permanance or beauty. They saw clearly that beauty and permanance subsisted in the nature of the material and the quality of its craftsmanship, rather than in such accidents of form as tesserae or granular particles. The fifteen years of experience, which this studio has had, with the practical application of modern methods to the making of mosaics was accepted by them as sufficient to prove that the age-old principles of craftsmanship had not been violated by the modern methods and that nothing had been lost of permanance or beauty.

The Department of Justice building is a modern office building to which its designers have given what I might call a facial expression characteristic of Greek architecture. It is done with a freedom enjoyed only by the few men whose knowledge, culture and experience permit them to design almost anything they wish without fear of a grave mistake. Such buildings are documents of present architectural trends. They clearly record both the vision of a new American architecture and an admiration for the beautiful conventions of a past civilization. Concrete made this building possible yet it finds no place in its appearance except where the mosaic concrete ceilings liberate this great material from hidden service and accord it a full participation in architectural expression.

The specification prepared by the architects provided that mosaic concrete should be used in the ceilings of the open passages from the streets to the courts, of the entries, and of the loggia. These ceilings are as fully subject to exterior exposure as ceilings can be. They are subject to all natural changes of temperature, to dust from the streets, and during the humid months they are almost constantly wet. The decoration of these ceilings with permanent color could be satisfactorily done with but few materials. Colors carried in vehicles affected by heat and cold, by water or dust or smoke could not be used. Only such materials as natural stone of fast color, ceramics or vitreous enamels were able to withstand such exposure. These materials used in the form of mosaics were known to be permanent, to be under close technical control, and to afford a wide range of beautiful colors.

Mosaic has always been held in high regard because of its permanance and beauty. The real and practical evidence of its great popularity is found in the fact, that, throughout the so-called dark ages, the most difficult period of our civilization, mosaic work contined to be done.

Mosaic requires hard and tedious labor from highly skilled craftsmen. It takes a long time to put together so great a number of tiny stones. Naturally, therefore as skilled workmen received higher wages the cost of mosaic increased. The cost continued to increase until now mosaic made in the old way is so expensive that it can be purchased only by the very wealthy, but in spite of its great cost every one loves its beauty and is fascinated by the many bits of colored stone forming its surface. The desire to use it for the decoration of buildings is just as strong as ever.

Sometimes we admire a beautiful flower which will not grow in our country or the fruit of a tree which is not hardy. In such cases we graft the plant desired onto the stem of some stronger but similar plant, which thrives in our environment. This, in a way, is what our American craftsmen have done with mosaic. Mosaic is a flower of the arts which does not thrive in our economic climate. But our craftsmen have grafted it onto concrete in such a way that mosaic is endowed with the economic vigor of concrete and again flourishes among modern arts.

This grafting was made possible by the marked similarity between mosaic and concrete. This similarity becomes immediately apparent when you remember that both materials are made by cementing together small pieces of stone. Mosaic is traditionally made by selecting pieces of beautifully colored stone or by making beautifully colored enamels and then breaking them by hand into small pieces. These pieces are then placed side by side and pressed one by one into a soft mortar. The whole process involves the handling of numberless pieces of solid material. On the other hand concrete is made by mixing small pieces of stone, crushed by machine, with cement and water and casting the mass in a mold. The modern mosaic used in the ceilings of the Department of Justice was made by a method similar to that by which concrete is made. Rare and beautiful materials were used but they are broken by machinery and not by hand, they were cast in a mold as a plastic material and not hand placed as small bits of a solid material.

This grafting of mosaic onto concrete has been of great mutual benefit. Concrete has contributed its strength and economy to the new material. Mosaic has not only contributed its permanent beauty but it has also freed concrete from the waste of expensive forms.

The beautiful pattern with which the mosaic-concrete ceilings of the Department of Justice building are decorated is integrally part of the pre-cast slabs made in our studio. These slabs were composed ex-

clusively of carefully selected white quartz, equally beautiful blue and yellow ceramics, black and red vitreous enamel and gold. These rare substances were crushed between great steel rolls and carefully screened to exact size. All dust was removed. No pigments were used, nothing but pieces of beautifully colored stone and stone-like materials, portland cement and water.

The reinforced structural concrete specified for the ceilings was of typical design. It differed in no way from concrete specified for use elsewhere in the building. However, the specification read:

> The mosaic finish and the structural concrete connected with it shall be one. They shall be made at the same time and by the same contractor. The finish shall in no way be applied to concrete previously constructed.

I tried to argue this requirement with the architects while the design was still in a state of flux and might have been modified in many ways. But Mr. Zantzinger would not consider an argument in favor of the application of the finish to a previously built structure. He was determined that they should be one. This was the die in which a useful idea was cast. Had Mr. Zantzinger weakened in his determination we would, I am convinced, have applied the decorations to the structure as we had done many times before. Then there would have been no adventure in the Department of Justice building, and we might have done the walls of our small prefabricated house with stucco instead of precast mosaic concrete slabs.

The ceilings of the entries of the Department of Justice building were not difficult. They were only 6 by 12 feet, and were cast in the studio in one piece. They were set in place.

The ceilings of the loggia were without much ornament and were not too large to be done in one day. Therefore typical wooden forms were made for them. The finish was applied to the face of the forms and the structural concrete placed upon it. Considerable care was exercised in placing the structural concrete that the surface materials might not be disturbed. Very close control of the water was necessary that the forms might be removed on the next day and the surface brushed with steel wire brushes to expose the aggregate evenly.

The ceilings of the two passages from the streets to the great court however presented quite a different problem. These ceilings were large, 60 by 60 ft. They were each to be supported by two great beams running in one direction. They also had intermediate beams running in the opposite direction to be bridged by slabs. Both the great beams and the areas between them were too large and too completely covered with ornament to be completed in one day.

The architects were most helpful, and offered to consider any design, the composition of which would divide the ceilings into parts that

could be done in one day. The structural requirements, however, were too difficult.

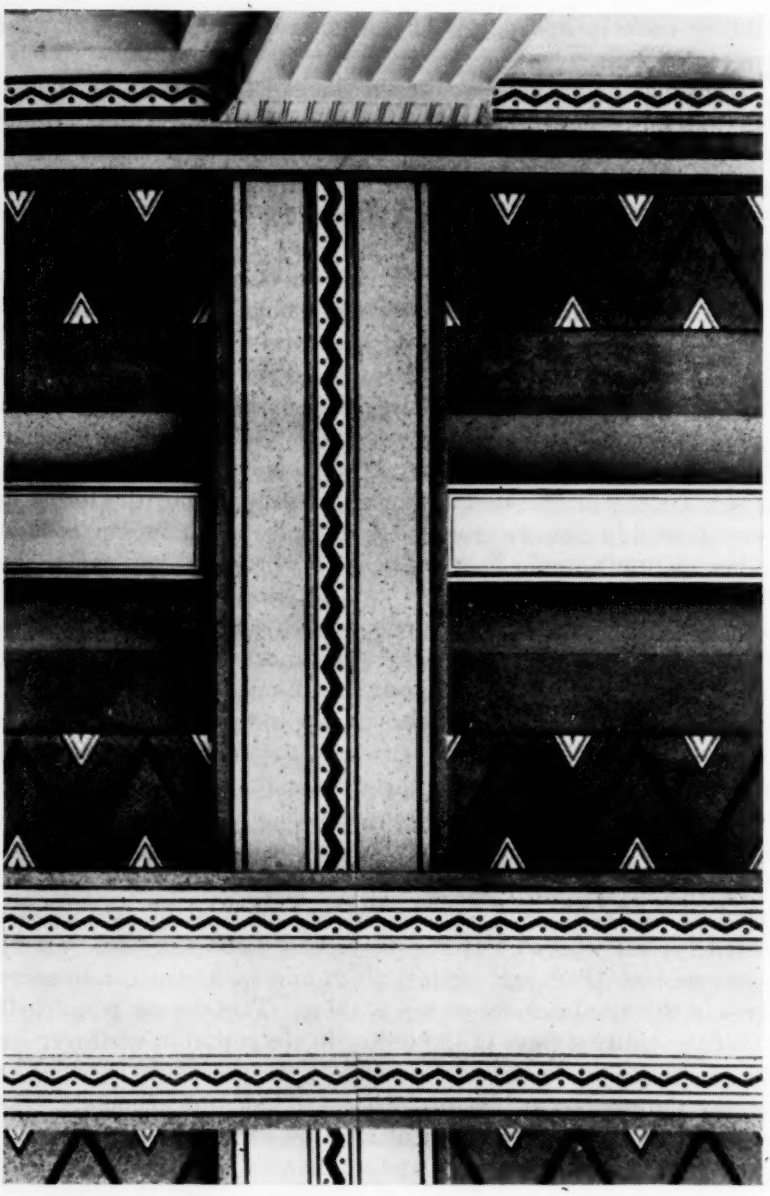
The beams for instance were 60 ft. long, 4 ft. wide and 4 ft. deep. They were ornamented on the soffit and on each side with continuous motifs the full length of the beam. Because of their size it was impossible to reconcile the element of time which is part of the process of making mosaic concrete and the time which is part of the process of placing structural concrete.

If the forms were allowed to remain in place until the structural concrete of the beam should be strong enough to support itself, the mosaic concrete of the surface, which had been placed first, would be too hard to permit the exposure of the aggregate by the brushing methods now in use. Not only would this mosaic concrete have had a lead of 24 hours in set over the structural concrete but its hardening would have been accelerated by the temperatures developed within the structural concrete itself. If, on the other hand, the forms had been removed in time to expose the aggregate in the mosaic concrete, the beams surely would have failed. This problem, you can believe, received close attention.

Since the beginning of our experience with concrete we have precast the more complicated parts of our work and have set them in place. When at first we applied mosaic concrete in much the same manner as stucco, we made all polychrome ornaments in thin slabs about an inch thick. At the time we did not realize how important such thin precast slabs were in our work. But it is clear now that their importance has been increasing in the last ten years and therefore it was only reasonable that we should turn to them for a solution of the problem presented by these large ceilings.

Our first shop drawings made for the approval of the architect, showed typical wooden forms with precast slabs of mosaic concrete about one inch thick laid against them and with structural concrete placed in the usual manner on top of these. This scheme permitted us to cast the entire surface of the ceiling in the studio in whatever sized pieces seemed best to us and to complete the process of exposing and cleaning the aggregate before placing the structural concrete. This method allowed the reinforcement and the structural concrete to be placed in a leisurely manner and to harden thoroughly before the removal of all supporting forms.

But this plan was never executed. The shop drawings made it apparent that we were planning to build two systems of forming in one —namely, wooden forms to support the weight of the structural concrete and mosaic concrete forms for appearance. Then an idea came



BEST PERHAPS OF NUMEROUS ATTEMPTS TO PHOTOGRAPH MOSAIC CONCRETE CEILINGS OF U. S. DEPARTMENT OF JUSTICE BUILDING

This shows patterns on underside of slabs and beams but gives little notion of color and texture.

to us. If we were to strengthen the mosaic concrete slabs we might use them as forms to support the structural concrete while setting and thus discard the conventional wooden forms and supports. The idea opened up an entirely new approach to the problem and led to a very satisfactory solution.

The shop drawings were remade. The mosaic concrete slabs were increased in thickness from one to two inches. They were reinforced with electrically welded steel wire mesh designed for the load of structural concrete to be supported and for the handling to be undergone in the processes of prefabrication and shipment. Steel loops protruded from the backs of the slabs. These were so arranged that the slabs could be temporarily supported by hangars from the floor above so that the structural reinforcements could be threaded through the loops mechanically to attach the surface slab to the structure.

When this scheme of construction was submitted to the architects they approved it, but they increased the depths of the structural members upon the theory that the slabs might safely be considered as one half live load and one half dead load. However; I received the impression that this conservative action was prompted by lack of precedent and was taken in spite of the reasonable conviction that these precast slabs mechanically tied to the structural reinforcements and roughened on the back for bonding might just as well be considered as wholely part of the structural members.

And so the ceilings were built. The precast slabs were made in our studio and taken to the building and set in place. Into them as into a mold the structural concrete of the second floor was poured. A strange thing had been done. The decorated surface of a concrete structure was its mold. The concrete had, if I may use the figure, been poured into its own skin.

If at this point we may reminisce for a moment, it would seem that everything that has been done to improve the appearance of mosaic concrete also improved the strength and permanance of structural concrete. When we separated the aggregate into two sizes and recombined it to the greatest density practically possible, it was done to obtain the greatest possible value from the color of the aggregate and to reduce to a minimum the interference from the color of the cement. But the same procedure increased the density of the structural concrete and reduced permeability to a minimum. When we extracted water from mosaic concrete castings it was done to stiffen them so that they could be taken from the mold and surface treated before the cement had thoroughly set. In doing so we made a cement water paste which produced the strongest structural concrete. Therefore

mosaic concrete possesses with its beauty a high degree of impermeability and strength.

Strength is not nearly so important as impermeability. Comparatively weak concrete will last for a long time if protected so that water cannot pass through it.

The use of impervious precast concrete slabs as the forms into which structural concrete is cast, saves the cost of the usual wooden forms. At the same time, the slabs give a far more rigid form than any wooden frame work could produce.

By using slabs—with an integrated, ornamental surface—there is a double economy in the saving of the costs of forming and the costs of a subsequently imposed decorative surface. This process should contribute much to increasing the use of concrete for all forms of structures in which a water-tight or ornamental surface is a requisite. It promises revolutionary possibilities for bridges, as well as buildings.

Concrete made by the best methods has great resistance to the passage of water. But, although the best methods have been published to the industry by the American Concrete Institute they have not yet been put into general use in the field. Many excuses have been offered for known defects in current practice. Most of them are economic and not technical. None of them is convincing to me: I can not recognize an excuse for the manufacture of concrete that is less than the best which the art affords.

Concrete properly made is resistant to water and therefore may be called a permanent material. I do not mean that well made concrete is everlasting. No building material is that. But, rather, it is meant that well made concrete is as permanent as the most permanent building materials known to the industry, that a well built concrete building will outlast its usefulness, and will require very little maintenance. This, and not an initial low cost of manufacture, is to me the justification for the use of concrete.

I will admit that it is easier at this time to produce concrete of superlative quality in the factory than in the field. And I sincerely believe that prefabricated mosaic concrete slabs such as are made in our studio are as close to superlative quality as the present state of the art permits. They are therefore as weatherproof and as permanent as any comparable building material and when assembled as forms for structural concrete they form a skin which is integral with the structure and which protects it from the elements, thus giving it both beauty and integrity.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for Sept.-Oct. 1935 (Proceedings, Vol. 32). Discussion should be available to the Secretary by Aug. 1, 1935.*

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's corps of Reviewers.

Place and function of steel in concrete columns

LIEUT. COMDR. BEN MOREELL, Bureau of Yards and Docks, U. S. Navy, *Engineering News-Record*, Vol. 114, No. 12, March 21, 1935, p. 410. Reviewed by N. M. NEWMARK

Commander Moreell discusses the type of concrete column composed of a plain concrete shaft encased in a steel shell as proposed by W. S. Lohr in *Eng. News-Record*, Dec. 13, 1934, p. 760 (reviewed in JOURNAL, A. C. I., Jan.-Feb. 1935, p. 336). A brief review of the history of laterally restrained concrete compression members is given and several objections to the Lohr method are presented in detail.

Designing composite steel and concrete bridge slabs

SHORTRIDGE HARDESTY, *Engineering News-Record*, Vol. 114, No. 14, April 4, 1935, p. 484. Reviewed by N. M. NEWMARK

Light weight roadway slabs of the concrete-filled steel-grid type weighing about 45 lb. per sq. ft. have been used in New Jersey for two bascule bridges. The article describes tests on three such slabs, one unfilled, one filled with concrete but with the bond destroyed by a paraffin coating, and the third filled as in actual practice. An analysis of the test results is given and recommendations for the design of the composite units are made, based on the tests and analysis.

Vibration of concrete

RIKARD V. FROST and VIKTOR BAHRNER, *Betong*, No. 1, 1935, p. 1. Reviewed by INGE LYSE

A discussion of the advantages and disadvantages of placing concrete by means of vibrators. Results obtained in different countries are studied and descriptions of a number of the different vibrators produced in the United States, France, and Sweden are given. Illustrations of the various types of vibrators as well as of the different job conditions are presented. Recommendations are given regarding speed of vibration, gradation of aggregates, external and internal vibrations, water content in concrete mix and form requirements.

Lightweight slag concrete

MEYER, *Zement*, No. 10, March 7, 1935. Reviewed by INGE LYSE

Water-cooled porous slag was used as aggregates in concrete for light weight. Description of method of production and information on unit weight of various particle s izes of the slag are presented. The relationship between amount of mixing

water and strength of the light weight concrete is discussed, and the importance of sufficient fines in the slag. The specific gravity of the concrete containing porous slag as aggregate varied from 1.20 to 1.25. The results are not compared with similar tests of ordinary concrete.

1850-year old concrete and its use as artificial stone

R. GRUN, *Zement*, No. 15, April 11, 1935, p. 232. Reviewed by INGE LYSE

Describes the conduit, 50 miles long, constructed as early as 80 to 90 A.D., by the Romans for the water supply of the city Köln in Germany. The conduit must have been in use for more than 400 years. After it was destroyed, much of its concrete was used in the construction of castles and walls. The concrete shows high quality, has an average compressive strength of about 1600 p. s. i. and shows little evidence of disintegration. The concrete consists of hydraulic lime as binding material with quartz sand and crushed limestone as aggregate. The inner surface of the conduit is lined with a plaster which still binds the concrete.

The large wind tunnel at Chalais-Meudon

M. LEMAREC, *L'Entreprise Francaise*, Vol. 5, No. 51, p. 19. Reviewed by P. H. BATES

The author gives a very interesting but not too technical description of this very large concrete wind tunnel erected for studying aerodynamical specimens as large as full-sized aeroplanes. The paper is very well illustrated with photographs showing the completed structure as well as phases of the work as the construction progressed. The large size of the structure can be realized from the figures showing the axes of the mouth of the tunnel which is in the form of an ellipse. The horizontal axis is approximately 25 metres and the vertical axis 17 metres. The structure comprises not only the wind tunnel but the necessary accessory laboratories, shops, offices, etc.

New type seawall built for subsiding lake shore in Venezuela

J. J. COLLINS (Project Manager, Raymond Concrete Pile Co.), *Engineering News-Record*, Vol. 114, No. 12, March 21, 1935, p. 405. Reviewed by N. M. NEWMARK

To prevent an extensive oil field development from flooding due to a subsidence of ground on the eastern shore of lake Maracaibo at Lagunillas, Venezuela, a continuous wall of reinforced-concrete sheet piles, 14,731 ft. long, was constructed. The concrete sheet pile design was chosen for its watertightness, resistance to corrosive lake waters, and flexibility as the ground is still settling at the approximate rate of 12 in. annually.

The wall is made up of 11½ by 24-in. sheet piles about 31 ft. in average length, tied together by a 24 x 27-in. concrete back beam and braced by inclined composite brace piles. The wall has survived a slight earthquake and almost a year of service with a land subsidence of about 1 ft., and has weathered two severe storms without any apparent damage.

Concreting plant and methods on Mississippi river locks

J. B. ALEXANDER and SAM COMESS, *Engineering News-Record*, Vol. 114, No. 16, April 18, 1935, p. 543. Reviewed by N. M. NEWMARK

This article describes the plant and procedure employed in mixing and placing 615,000 cu. yd. of concrete in seven locks and one dam in the Rock Island engineer district. A tabulation of the composition and strength, and of the manner of batching, mixing, transporting, and distributing of the concrete, as well as the production rates on the various jobs, are given. Dry concrete (1-in. slump) was used, and compacted by high-frequency internal vibrators. Because of the variable physical characteristics of the materials used on the different jobs no uniform procedure in vibrating

and spading could be applied to all projects. On some of the jobs difficulty was encountered in accumulation of free water on the surface of the concrete. The curing processes and the special precautions in handling and curing concrete at low temperatures are described.

Uniformity in grain sizes in cement

GEORG ROTHFUCHS, *Tonindustrie-Zeitung*, March 25, 1935, No. 25, p. 318. Reviewed by INGE LYSE

A discussion of the gradation which will give maximum density of material consisting of different sizes. The natural gradation of aggregates will tend to give a maximum density combination and this gradation may be expressed by the formula:

$$p = \frac{\sqrt{d} - \sqrt{d_{min}}}{\sqrt{D} - \sqrt{d_{min}}}$$

where p = per cent passing sieve opening d
D = maximum grain size
d_{min} = minimum grain size

Plotting the amount of material retained on each sieve against the square root of the sieve opening gives a straight line relation.

An analysis of several cements showed gradation well in agreement with the above formula.

Inclined hangers impart slenderness to ribs of 470 ft. concrete arch

A. E. BRETTING (Chief Engineer, Christiani and Nielsen, Copenhagen), *Engineering News-Record*, Vol. 114, No. 17, April 25, 1935, p. 577. Reviewed by N. M. NEWMARK

A reinforced concrete through-arch highway bridge of 470 ft. span over the Lot river at Castelmoron in southern France is described. The bridge provides an 18-ft. roadway and two 3-ft. sidewalks, and was designed for a uniform load of 100 lb. per sq. ft., or for 4, 16-ton trucks, and a wind pressure of 50 lb. per sq. ft. Maximum stresses were fixed at 1140 p. s. i., although the precast core section of the arch ribs (used to reduce the load on the centering) was stressed to about 1900 p. s. i. The Freyssinet method of decentering was used.

The rib is hinged at its intersection with the roadway which acts as a tie, and a third rib hinge is placed at one abutment. Inclined steel hangars support the roadway, so that the bridge is essentially a parabolic-chord truss with high secondary stresses. The feature of the inclined ties is in accordance with a system patented by O. F. Nielsen who has described this type of construction in an article entitled "Bogenträger Mit Schräg Gestellten Hängestangen," Publications, Int. Assoen. for Bridge and Structural Engineering, 1932, p. 355-364.

Effect of method of manufacture on the properties of cements

G. HAEGERMANN, *Tagung des Vereins Deutscher Portland-Cement-Fabrikanten*, Sept. 5, 1934, p. 23. Reviewed by INGE LYSE

This is a timely contribution to our knowledge of what influences the properties of portland cement. The four main contributors to the properties of the cement are: 1. the raw material, 2. chemical composition, 3. the burning, and 4. the fineness to which the clinker is ground. The fineness to which the raw material is ground is also important for the final quality of the cement. Data are presented showing a marked increase in both compressive and tensile strength with increased fineness of raw material. The most important chemical compositions are the lime and silica contents.

The old rule of high lime content giving high strength still holds true within ordinary limits. However, high lime content requires high burning temperatures for the clinker. The fusion heat for the clinker is very important for the quality of the cement. Increasing the temperature from 1400°C to 1450°C produced an appreciable increase in the strength. Although the heat was somewhat dependent upon the type of raw material used, the importance of "sharp" burning for securing the optimum quality of the cement should be realized. The fineness to which the clinker was ground also had large influence on the quality of the cement.

Twelve diagrams present the relationships found for the various effects upon the cement.

Effect of calcium chloride on portland cements and concretes

PAUL RAPP, Research Paper (RP 782), *National Bureau of Standards Journal of Research*, Vol. 14, April 1935. AUTHOR'S ABSTRACT

To study the effect of the addition of calcium chloride on portland cements of widely varying composition, and on portland-cement concretes, a carefully controlled laboratory investigation was conducted. Such measurements as heats of hydration, setting times, consistencies and the strengths at various ages under several curing temperatures were made.

The addition of calcium chloride appears to increase somewhat the heat contributed at the end of 24 hours by dicalcium silicate and tetracalcium alumino-ferrite and to decrease the heat contributed by tricalcium aluminate. Calcium chloride has but little effect on the heat contributed by tricalcium silicate. Calcium chloride increases the rate at which the heat is evolved from all cements. The strength contributed by the two silicate compounds in cement was increased and the strength contributed by tricalcium aluminate was decreased by the addition of calcium chloride. Calcium chloride decreased the time of set of 11 commercial cements and increased the flow and the strength of the resulting concrete. The physical properties of the experimental cements were similarly affected by the addition of calcium chloride. The optimum amounts to be added for different types of cement and at various curing temperatures are indicated.

Some notes on the manufacture of portland cement

KARL E. DORSCH, *La Revue des Materiaux de Construction et de Travaux Publics*, No. 303 to 306, Dec. 1934 to March 1935 inclusive Reviewed by P. H. BATES

The title is significant of the nature of the articles presented by Dorsch. The first number is given over to a general discussion of the economies of cement manufacturing as limited by prevailing sales price conditions and plant organization, and to the question of cement quality. He shows that there are so many variables that no general rules may be advanced. So far as quality is concerned, he believes that it depends upon the usage to which the cement will be placed and not upon its satisfactorily meeting more or less closely certain standard requirements.

In the second article the matter of plant supervision is presented. The conclusions are that both management of the plant and the laboratory should be in the hands of well educated and well trained technicians. The plant and the laboratory should be independently supervised.

The third installment is given over to an amplification of plant, and more particularly laboratory operating details; while the last number is confined to the solution of a number of problems which may arise in plant operation. Some of these are

difficulties encountered in securing the sought for raw mixtures, how to overcome expanding cements, too quick or too slow setting cements, poor strength, etc.

This series of papers will be of much interest to cement plant operators in acquainting them with foreign viewpoints in plant operation.

Computations of reinforced concrete ribbed chimneys

M. CHALOS, *Le Genie Civil*, Vol. CVI, No. 11, p. 251-254. Reviewed by R. L. BERTIN

Calls attention to the necessity of close approximation of the actual stresses to which high chimneys are subjected.

The general designs are based on the method of Saliger or the graphs of Mesnager and take account merely of the moments resulting from wind pressure, the footing slab being considered anchored to the shaft of the chimney.

Such methods and assumptions are sufficiently accurate for chimneys of constant taper and moderate height, solidly anchored in the ground. They may, however, lead to serious errors when applied to very tall chimneys.

The author points out that wind pressure increases with the height above the ground; that the total pressure is the product of the unit pressure by the diametral surface, modified by a factor which varies with the shape of the shaft.

In the absence of precise tests, the pressure for chimneys of considerable height should not be less than 150 kg. per sq. meter on the diametral surface.

In evaluating the weight of the chimney on which its stability depends, the true unit weight of the concrete must not be exceeded. The moment computation must include for tall and slender chimneys the moments resulting from the deflection of the shaft and foundation slab, as well as the yield of the soil.

Equations are given for computing the resisting moment accurately which, when based on the elastic limit of the steel or the ultimate compressing resistance of the concrete, should not be less than twice the total bending moment.

In proportioning the footing slab, the resultant of all forces should fall within the kern section and the soil reaction should be such as to preclude measurable settlement.

Equations for computing these slabs are given in detail.

Finally, the use of Timochenko's critical slender ratio equation is recommended as a check against buckling.

Effect of granulometric composition of cement on the properties of pastes, mortars and concretes

J. ARTHUR SWENSON, LACEY A. WAGNER and GEORGE L. PIGMAN, Research Paper (RP 777), *National Bureau of Standards Journal of Research*, Vol. 14, April 1935. AUTHORS' ABSTRACT

Data obtained in studies of five laboratory-ground clinkers and one commercial cement are presented. Each of the ground clinkers was separated into the following five nominally sized fractions: 0 to 7, 7 to 22, 22 to 35, 35 to 55, and > 55 microns. The commercial cement was separated into only four fractions, no separation being made of the material coarser than 35 microns. Tests were made on the individual fractions as well as on four cements prepared by blending the fractions in various proportions. The specific surfaces of these blended cements ranged from 1,350 to 3,300 sq. cm. per g. It was found that the 0-7 micron material is very valuable because of the plastic qualities which it confers upon the concrete mixes and also because of the high contribution which it makes to the early strength. The other four fractions were found lacking in plasticity and the ability to hold water, and the rate of strength development decreased with increasing grain size. The contribution of the 0-7 micron fraction to the strengths of the blended cements was calculated by an

algebraic method based on the assumption that the contribution of any fraction to the strength of a blended cement is equal to the product of the decimal part of the blend composed of that fraction and the strength of the fraction when tested by itself. The values thus calculated were found to be of the order which might be expected. The analysis of all the strength data tended to prove that the compressive strength of concrete is very nearly a direct function of the amount of cement which has reacted with water. When the strengths were plotted against specific surface, one line was obtained for the blended cements and another line for the individual fractions. When these same strengths were plotted against quantities which should at least be approximations of the amount of hydrous material, all of them fell on one line.

New French concrete regulations for work controlled by the department of Public Works

HENRY LOSSIER, *Le Genie Civil*, Vol. CVI, No. 13, March 30, 1935, p. 308-311.
Reviewed by R. L. BERTIN

These regulations, dated July 19, 1935, are intended to replace those of October 26, 1906. They were prepared by a special commission with which committees from the railroads and professional groups collaborated, among them the Association of Reinforced Concrete Constructors which in 1928 promulgated regulations taking account of the technical progress relative to the quality of the materials and the accuracy of the stress evaluations.

These new regulations follow in principle those of 1906 in that they are neither precise nor formal and leave ample room for individual initiative and innovation on the part of the engineers of the Department of Public Works for whom they are exclusively intended.

These regulations vary from those of 1906 as follows:

(a) Stresses are prescribed for three grades of steel.

(b) More definite instructions are given regarding shear reinforcement.

(c) The tensile bond and shear stresses of concrete are based on 1/5 of the ultimate extreme fibre tensile stress of the concrete.

(d) Bond stresses are varied depending upon the thickness of concrete around the bars, anchorage by means of hooks or bar extensions into neighboring masses of concrete.

(e) The permissible width of flange in T-beams is increased.

(f) Shrinkage consideration is made mandatory.

(g) Two-way slabs are liberalized.

(h) A reduction in the working stresses inversely proportional to their range of variation, not to exceed 25 per cent is provided.

(i) Care in the selection of materials and their installation is stressed.

The complete regulations are published in the March issue of *Annales des Ponts et Chaussees.*

New Beaujon hospital at Clichy (Seine)

PAUL CALFAS, *Le Genie Civil*, Vol. CVI, Feb. 2, 1935, p. 101-109 and Feb. 9, 1935, p. 132-136.
Reviewed by R. L. BERTIN

Describes in great detail the construction of this new hospital 177.74 meters long, 11.51 meters wide and 13 stories or 69.50 meters high. This main building is flanked with four wings 21.90 meters long and 10 meters wide, at the ends of which semicircular cantilevers are provided serving as solariums. The structure is of the skeleton

type, the frame of which is reinforced concrete. The description of the concrete plant, the job organization, and sequence of operation is of special interest.

The concrete was prepared in a central plant; two bins, one for sand and the other for gravel holding enough material for three days work were kept filled by trucks which brought the aggregate from a dock as needed.

The mixer was of the tandem drum type, selected to insure homogeneous concrete with a minimum mixing time. Concrete from the mixers was fed into a concrete pump capable of discharging 12 cubic meters of concrete per hour at a distance of 250 meters. The pump was capable of conveying the concrete to a height of 25 meters on the entire length of the building. Beyond 25 meters, another pump in tandem was used. The advantages claimed for this method of concrete distribution are cheapness of handling, regular delivery, which compels the workmen to keep up with the pump and a better homogeneity of the concrete.

A careful schedule of daily operations was prepared at the start of the work, based on what is claimed to be an entirely new procedure, which consisted in the organization of small gangs, the function of each one being to execute a certain phase of the work throughout the entire job, each gang following the other. Thus, the first gang would set the column steel followed by the second gang setting the column forms, followed by the third gang setting the beam forms, and so on. It is claimed that the advantage of this method lies in the dexterity acquired by the men doing just one thing which increases production and makes for more perfect execution, and the fact that the various gangs working on different parts of the job do not interfere with each other.

The concrete was compacted by means of pneumatic vibrators, compressed air being used also for cleaning the forms immediately before concreting. The compressed air was produced by three 25 h.p. compressors.

All care was taken for the protection and comfort of the workers. The openings and edges of the floor in course of construction were protected with guard rails and toe boards; the stairs, toilet facilities and a lunch counter followed one floor below that under construction.

A job laboratory was provided for testing the concrete materials, the mixture and the concrete specimens. The operation, maintenance and repair of the tools and machinery was taken care of by 25 mechanics. A total of 400 h.p. was required for the operation of all the machinery. The authorities claim a degree of perfection in the execution of this work never before attained.

Permeability of concrete

M. MARY (Engineer of Ponts et Chaussees), *Les Annales des Ponts et Chaussees*, Vol. II, No. VI, Nov.-Dec. 1934. Reviewed by B. MOREELL

This article continues an account of experiments conducted at the laboratory of Ponts et Chaussées, the first series of which were described in *Les Annales* for May-June 1933. Hollow concrete specimens were filled with water which was subjected to pressure. The conclusions drawn from the first series of tests are as follows:

(a) Concretes aged under water are much less permeable than concretes which have been allowed to age in air; the latter, however, become less permeable when subjected to water under pressure and tend to approach the condition of concrete aged under water.

(b) When concrete is subjected to the passage of a water saturated with bicarbonate of soda, there is formed an outer skin several millimeters in thickness, which reduces

the permeability. This arises from the fact that the free lime liberated by the setting action of the cement is precipitated by the bicarbonate.

(c) Water passing through the concrete becomes saturated with the free lime until it has removed an appreciable percentage of the weight of cement.

The second series of tests led to the following conclusions:

(a) For concretes aged in air, the permeability decreases with age; for concretes aged in water there was no appreciable improvement in water-tightness after the concrete was 8 days old.

(b) In investigating the effect of the granulometric composition of the aggregates on the permeability, it was found that the first requisite for an impermeable concrete is that it should be thoroughly plastic and workable. Concretes having sands conforming to Bolomey's grading curves are less permeable than those made with badly graded sands. In general, the permeability varied inversely as the strength, although there was one exception to this rule.

(c) Practically speaking, Kieselguhr used as an admixture had little influence on the density of the cement paste; but Kieselguhr did improve the plasticity of the concrete and, on the job, would probably effect an improvement in water-tightness by virtue of this influence.

(d) Experiments conducted with various types of slag-cement mixtures gave such diverse results that no definite conclusions as to relative values could be drawn. It is evident, however, that the characteristics of the cement have an important effect on permeability.

(e) Portland cement concretes aged in air show an appreciable loss in weight (up to about 10 per cent of the weight of cement) when water passes through the concrete. When the water is distilled, the decomposition continues indefinitely; with calcareous water it is finally arrested. When the concrete is aged under water, the decomposition is greatly decreased. Concretes made with the so-called "metallurgical cements" (slag-lime or slag-cement mixtures) showed very little decomposition by water passing through the concrete.

The author concludes with a warning that laboratory results cannot always be made the basis of practical application in the field. He stresses the importance of (a) workability, (b) the cement factor, (c) characteristics of the cement.

Exterior waterproofing materials

D. W. KESSLER, *National Bureau of Standards Journal of Research*, March 1935 (RP 771).

AUTHOR'S ABSTRACT

The clear waterproofing materials designed for application to the exterior faces of masonry buildings find a considerable field of use for the purpose of preventing the penetration of dampness through the walls. Heavy rainstorms accompanied by strong winds quite frequently cause enough moisture penetration through various types of masonry walls to injure the interior decorations. In such cases the moisture may penetrate through the pores of the masonry units, through the pores of the mortar joining the units or through openings which often occur between the units and mortar. The exterior waterproofings are intended to seal the pores of the masonry at the exposed face without materially changing the natural appearance. Megascopic cracks usually cannot be sealed with a waterproofing treatment of this nature.

To supply the demand for information on the reliability of such treatments an investigation was started at the National Bureau of Standards in 1921. Points of chief concern were (1) the effectiveness of the different types of treatment in sealing

the pores of masonry and (2) the durability of the treatments. The initial effectiveness can be readily determined by treating specimens of the masonry units and measuring the reduction of absorption. The durability can be determined only by exposure tests which require a number of years.

Samples of the various trade products were collected, classified and applied to various masonry materials. Such processes of a nonproprietary nature as have been used or suggested were also included in the studies. It was found that some types of treatment were of little value from the start and others which initially gave good waterproofing values deteriorated rapidly. However, some types have quite consistently shown high waterproofing values over exposure periods of 10 to 12 years.

Wax types were indicated to be the most durable, but they have the undesirable property of producing discolorations on most masonry materials. The insoluble soap type produces no appreciable discoloration, but it is not very durable. Fairly satisfactory results were indicated for the thinned fatty oils and better lasting qualities were indicated for thinned fatty oils with a high melting point paraffin in solution; however, with such types it seems necessary to adapt the consistency of the treatment to the pore structure of the particular masonry under consideration. Treatments such as normal varnishes of thin consistency, lacquers and wax emulsions which produce a film were not found to be very satisfactory. Treatments of two reacting solutions which produce insoluble precipitates, and also treatments which are intended to react with the masonry were not found to be very effective.

Some information was obtained, incidentally, on the preservative value of waterproofing treatments and the adaptability of the treatments to pore structure. The more effective and durable types retarded the solvent action of rain water on calcareous materials, and increased the resistance of masonry material to frost action; they also appeared to have some value in overcoming the crystallizing action of salts which is a common cause of masonry decay.

It was found that waterproofing treatments are not usually effective to the same degree on different types of pore structure. A particular treatment that gives good results on one masonry material may give poor results on a different type of masonry. In general the more porous materials are easiest to waterproof and materials having very fine pores are difficult to seal.

New researches on the shrinkage of cements

M. Edmond Marcotte (Chief of Section of Physical and Mechanical Tests of the Laboratories of L'Ecole Nationale des Ponts et Chaussees), *Les Annales des Ponts et Chaussees*, Vol. II, No. VI, Nov.-Dec. 1934. Reviewed by B. Moreell

Extended research on the shrinkage of cements and mortars has been carried on at the laboratory of L'Ecole Nationale des Ponts et Chaussées for a number of years. A first partial report was published in *Les Annales*, Vol. III, 1932. Since that time experiments have been conducted to obtain more information on the phenomena incident to the shrinkage in air and the expansion in water of cements. The specimens were made of neat cement paste and were 13 mm square with lengths of 22 cm and 80 cm. Measurements of changes in length were made by means of an amplifying apparatus and by micrometers.

Curves are given to indicate the shrinkage in air and the expansion in water. An interesting observation is that when the specimen is immersed in water immediately after fabrication, its ultimate shrinkage is considerably more than that of corresponding specimens which are aged only in air. This is attributed to the fact that while the specimen is being aged in water, some of the free lime is leached out. When

specimens are aged in air, it is believed that the free lime of the cement combines with the carbon dioxide of the air, which results in a diminution of the shrinkage. This might explain the small shrinkage noted when appreciable puzzolonic action takes place in concrete.

An interesting phenomenon is that the shrinkage of the specimen is not in direct proportion to the loss of weight due to evaporation of the contained water.

Experiments were performed to determine the effect on shrinkage of temperature variations and humidity conditions, the relation between shrinkage and loss of weight, the stresses caused in the specimen by shrinkage, and the effect of various liquid and powdered admixtures on shrinkage.

A very interesting feature of these tests is the determination by photo-elastic measurements of the stress caused by shrinkage. Small glass discs, 16 mm in diameter, were imbedded in the specimen. These discs were pierced by a hole and the principal stresses were measured directly at the edge of the hole.

It is claimed that by this method the principal stresses are determined directly. Methods formerly used, wherein the differences and the sums of the principal stresses are measured, would not have been applicable in this case, as the differences of the principal stresses would have been zero. Compressive stresses as high as 1400 p. s. i. were registered. The compressive stress was rapidly transformed to a tensile stress when the specimen was submerged.

By means of these stress measurements, the variation in the modulus of elasticity was determined. The author concludes that "there is in reality no such thing as a modulus of elasticity for cement but, instead, a variable relation between the unit stress and the unit strain."

The author issues a word of caution with respect to the bond of reinforcement in concrete which is submerged. He believes that his experiments indicate that the bond on such bars is very much reduced, and strongly recommends the use of deformed bars in accordance with American practice. The French practice is to use all plain bars.

Another interesting conclusion is that all of the admixtures, both powdered and liquid, result in an increase in shrinkage.

The article is a progress report and will be followed by others. Those who are interested in the study of volume changes of cements would profit by a careful study of this report.

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PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

VOLUME 31—1935

From JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 6, Sept.-Oct., 1934 to May-June 1935.

This Index combines references to:

Original contributions to the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE—papers, reports, discussions and their authors.

Current Reviews, published by the Institute, of significant contributions to the literature, originating elsewhere. Each Index reference to a Review is so designated.

In general, important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN. "Surface treatment" and other subjects classified under this head, are indented.

Key words to important subjects appear, in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Blast furnace slag," each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names (except in references to papers *Reviewed*) appear in proper alphabetical sequence with the subjects, with references to their contributions.

Specific data on Beams are so indexed by reference to ENGINEERING DESIGN or TESTS OF MEMBERS AND COMPLETED STRUCTURES thus avoiding an oversight by the searcher of important allied data.

The readiest use may be made of this index by gaining some familiarity with the thirty main classifications.

The searcher for information on design and proportioning of concrete mixtures will look for MIXTURES and refer to subheads: "Design" and "Proportions." Field control methods will be found under TESTS AND FIELD CONTROL.

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